CONTINUOUSLY REINFORCED CONCRETE PAVEMENTS
A REPORT OF THE STUDY GROUP

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CONTINUOUSLY REINFORCED CONCRETE PAVEMENTS
A REPORT OF THE STUDY GROUP

ABSTRACT
Continuously reinforced concrete pavements are widely used in the USA and also in Belgium. A Study Group of British engineers has visited both these countries to study the design, construction and performance of this type of pavement.

The concept of continuously reinforced concrete pavements is that cracks will occur at intervals of between 1.5 and 2.5 metres but that these cracks will be kept tightly closed by the reinforcement thus maintaining load transfer across the cracks by aggregate interlock.

No special construction equipment is necessary for continuously reinforced concrete pavements and there is no restriction on length other than a minimum of 150 m. The Study Group have concluded that for heavily trafficked roads the slab thickness for a continuously reinforced concrete pavement may be 30 mm less than that required for a jointed concrete pavement; this is associated with a steel percentage of 0.6.

The initial cost of a continuously reinforced concrete pavement is likely to be greater than that of a jointed concrete pavement for the same traffic conditions. Maintenance requirements are less and for heavily trafficked urban roads where delay costs due to maintenance operations may be high there could be economic advantages in the use of continuously reinforced concrete pavements.

1. INTRODUCTION
In the USA continuously reinforced concrete pavements (referred to hereafter as CRCP) have been in use for over 35 years and, following a slow start with experimental projects, the usage of CRCP has rapidly increased such that by 1972 thirty States had completed 16,500 km of equivalent two-lane pavement 7.3 m wide on the Interstate system. In the last five years about 8,050 km of equivalent two-lane pavement has been laid; the growth of CRCP in the USA is illustrated in Fig 1. Belgium constructed its first experimental length of CRCP in 1950 which was followed by other experimental roads until in 1970 a large programme of construction was initiated; during 1972 about 200 km of CRCP was being laid on autoroutes (about 600 km of
equivalent two-lane pavement). Other continental countries are investigating CRCP and the length laid of this form of construction is likely to increase in the near future.

At present in Great Britain CRCP is only permitted as a base to a bituminous running surface. Because of the widespread use elsewhere of CRCP as the running surface, the possibility of its use in this country needed evaluating. To this end a Study Group was formed which made a two week visit to the United States. CRCP roads were seen in six States and discussions were held with engineers from five States; meetings also took place with the Continuously Reinforced Pavement Group (CRPG), the Portland Cement Association (PCA) and the Federal Highway Administration. Some members of the Group subsequently visited Belgium to see work there. Members of the Study Group were:

Mr A E Burks  Consulting Engineer representing the Reinforcement Manufacturer’s Association (Leader of the Study Group)
Mr G B Colbridge  Constrado (Constructional Steel Research and Development Organisation)
Mr J M Gregory  Pavement Design Division, Transport and Road Research Laboratory, Department of the Environment
Mr J R Lake  Engineering Intelligence Division, Department of the Environment
Mr J C Lucas  Research and Development Division, British Rail Technical Centre
Mr A Pink  Advisory Division, Cement and Concrete Association
Mr R J Slowe  The BRC Engineering Co Ltd

The information obtained from the visits and from a study of literature is summarised in this report together with the conclusions reached and the recommendations based on these conclusions. A more detailed review is contained in Appendix 1 and the various items in the main report are cross referenced to Appendix 1, where this is likely to be helpful.

The States visited and the engineers met by the Group are listed in Appendix 2; all these States use CRCP and because of time limitations those States which do not use CRCP on a large scale (notably California) were not visited. Two short visits were made by some members of the Study Group to Belgium. Here conventional jointed pavement hereafter referred to as (JCP) and CRCP are being used and both forms of construction were seen. The Belgian engineers who were met are listed in Appendix 3 along with the sites inspected.

1.1 Basic concepts of Continuously Reinforced Concrete Pavements

The earliest concrete roads were unreinforced and were laid without joints. Because of the closely spaced transverse cracks which developed, transverse joints allowing some longitudinal movement were introduced. It was subsequently found that the spacing of the joints could be increased if the slabs were reinforced with steel. Experience has shown that in unreinforced concrete the joints need to be at a spacing of about 5 m and that this spacing can be increased to 20–30 m if reinforcement of up to 0.3 per cent of the cross-section
of the slab is laid in the longitudinal direction. In such concentrations, the steel does not influence crack formation; its function is to hold together the cracks which form, and to preserve granular interlock. To do this successfully over a long period the steel must be strong enough to resist the tensile stress which will develop at the cracks.

Experience has further shown that if the concentration of steel is increased well above that normally used in jointed reinforced concrete, the presence of the steel helps to limit crack formation as well as to hold tightly together cracks which do occur. It then becomes feasible to use continuously reinforced concrete as an alternative to plain and normally reinforced jointed concrete. There is some evidence to suggest that continuously reinforced pavements can be laid thinner than other concrete slabs carrying the same traffic, possibly due to the absence of load-induced stress concentrations near the joints.

There is now no doubt that it is possible to construct successfully all three forms of concrete pavement. Evidence from major concrete roads laid in Britain over the past 3 years shows that the first cost of plain concrete roads is less than that of the equivalent jointed reinforced concrete road, ie the cost of the additional joints is less than the cost of the reinforcement. The long-term cost of joint maintenance appears to be much the same; the smaller openings of frequent joints being no more costly to deal with than the larger joint movements in reinforced roads. The question posed by continuously reinforced concrete is whether the complete absence of joint maintenance during the life of the pavement, and possible economies in slab thickness, will outweigh the additional cost of the heavy reinforcement. To gain information helping to answer this question was a main purpose of the visit.

2. BEHAVIOUR OF CONTINUOUSLY REINFORCED CONCRETE PAVEMENTS

The main factors which influence the performance of Continuously Reinforced Concrete Pavements are examined below. Most of these also influence the other forms of concrete pavement to a greater or lesser degree.

2.1 Sub-base; support and frictional properties (See Appendix 1, 9.1, 10.1.1, 10.2.1 and 12.1)

The sub-base provides some structural support for the concrete slab and may play an important part in facilitating construction. In plain concrete, where no cracking is permissible, sliding of the concrete over the sub-base reduces restraint stresses in the slab, and low friction such as is afforded by a sliding layer is desirable. In jointed reinforced concrete some cracking is expected and medium friction is desirable to limit the movements occurring at joints. In CRCP, uniform and frequent cracking is a requirement to limit the tensile stress developed in the steel. This condition is best achieved by a uniformly rough interface between the sub-base and the concrete.

In America CRCP is normally laid directly on a granular sub-base stabilized with cement or lime. In Belgium a thin bituminous layer is placed beneath the concrete. Both of these methods appear to give the required frictional properties.

2.2 Reinforcement and concrete strength (See Appendix 1, 9.2, 9.3, 10.1.2.2, 10.1.3, 10.2.2.2 and 10.2.3)

Reinforcement weight, concrete strength and crack spacing are all closely interrelated in CRCP. As the strength of the concrete is increased, the crack spacing increases (for constant interface friction) and the
greater is the concentration of steel necessary to hold the cracks closed. In the USA, steel ranging in cross-section from 0.3 to 1.0 per cent has been used, deformed bar being more effective in controlling cracking than plain round steel. Experiments to determine the required percentage of steel have also been conducted in Belgium. Currently 0.6 per cent of steel appears to be the standard in the American States using this form of construction. In Belgium, where the concrete strength specified is higher than in the USA, 0.85 per cent of steel is generally used. The Study Group has concluded that 0.6 per cent of steel should be satisfactory for British conditions, provided the concrete strength does not greatly exceed the minimum value of 28 MN/m² (cube strength) at 28 days.

2.3 Crack spacing and crack width (See Appendix 1, 10.1.3.1)

The spacing of cracks is influenced by the percentage of steel in CRCP as well as the strength of the concrete. The rate of crack development is high during the early life of the pavement; after about three years the amount of new cracking is usually very small. As with JCP, cracking is greatly affected by the conditions appertaining at the time of construction, but crack spacing may also be influenced by the spacing of the transverse bars. Concrete strength has an effect on crack spacing, the spacing increasing with concrete strength when other factors are constant. This is most noticeable when comparing American and Belgian CRCP roads, the latter being constructed with a much higher strength concrete and having a much greater crack spacing. The optimum crack spacing appears to be between 1.5 and 2.5 m. (Plates 1 and 2.)

Crack width and crack spacing are interrelated. For the crack spacing given above the cracks will be sufficiently narrow to preserve granular interlock and prevent ingress of water. In some of the earlier Belgian construction, wide cracks resulted in shear failure of the reinforcement and this led to faulting. Work is in hand in the USA to determine permissible crack widths.

2.4 Slab thickness (See Appendix 1, 9.2, 10.1.2.1, and 10.2.2.1)

Recommendations for slab thickness were based on empirical data; subsequent theoretical studies, together with a limited number of loading tests, confirmed the empirical conclusion that for the same life a CRCP road need not be as thick as a corresponding JCP road. The extent of the thickness reduction varies from one state highway authority to another. The most widely used relationship in the USA is that 203 mm of CRCP is equivalent to 254 mm of JCP, but many American engineers are now questioning the validity of this equivalence. On the evidence acquired by the Study Group a reasonable reduction for roads carrying more than 11 million standard (8,200 kg) axles during the design life is 30 mm for UK conditions.

2.5 Effect of traffic on CRCP (See Appendix 1, 10.1.4)

The effect of traffic are the same for CRCP as for JCP and most failures have been in the heavily trafficked lanes. While it has been previously mentioned that cracking is primarily controlled by the climatic conditions at the time of construction and by the reinforcement percentage it is believed that traffic loadings cause the cracks to open slightly and any deficiencies in the concrete or the steel are then accentuated.

2.6 Maintenance (See Appendix 1, 10.1.6 and 10.2.5)

Both American and Belgian engineers stressed the reduced maintenance requirement of CRCP over other forms of construction; the life of the texture is assumed to be similar to that of JCP.
Failures usually occur in sections where the concrete is inadequately compacted or where there is insufficient overlap of the longitudinal steel and generally involve very closely spaced cracks often with bifurcation.

When it becomes necessary to overlay CRCP, either for strength or surface deficiencies, the normal overlay techniques may be used. The performance of a bituminous overlay on CRCP is likely to be better than on JCP because the movements at the cracks are significantly less than at the joints and cracks in JCP.

### 2.7 Surface regularity (See Appendix 1, 10.1.7 and 10.2.6)

Comparison of the rides obtained on CRCP and JCP roads in the USA and Belgium show the former to give a better ride. The fine cracks in the CRCP road do not affect the ride (nor are they visible to the road user) and with a total absence of transverse joints (other than end of day joints) it is to be expected that the riding quality of CRCP could be better than that of JCP.

### 2.8 Special ground conditions

Roads of CRCP have performed well when used in areas where sub-soil conditions are poor, eg over expansive clay soils. The pavement structure of a CRCP road is more flexible than that of a JCP road and this property supports the conclusion that CRCP could be successfully employed in areas where differential settlement may occur.

### 2.9 Length and geometric considerations

There is no restriction on the length of CRCP which can be constructed as a single slab other than a minimum which is of the order of 150 m. Present standards for horizontal and vertical curves of motorways and heavily trafficked trunk roads do not preclude the use of CRCP.

### 2.10 Overlays

During their visits the Study Group saw several CRC overlays; most of these were laid over existing concrete roads but one example of an overlaid flexible road was also seen. The minimum thickness of overlay which has been used is 152 mm and in all cases the performance appeared successful.

### 3. CONSTRUCTION OF CRCP

During their visit to the United States the Study Group confined themselves almost exclusively to the examination of the design and performance aspects of CRCP. Much information about construction techniques was however obtained during the discussions with American engineers and from published literature and films. In Belgium those members of the Study Group who visited that country spent part of their time looking at CRCP construction in progress.

The main advantage of CRCP over JCP from the construction aspect lies in the almost complete absence of transverse joints. However care is necessary in the layout of the reinforcement and in the connections between the longitudinal bars; consequently less work has to be done behind the paver or paving train but more work is necessary ahead of the laying equipment. The Study Group are of the opinion that construction of CRCP would involve a shorter learning period than has been the case with some of the new techniques associated with JCP, particularly in relation to joint construction. However, it is essential that in CRCP the
sub-base and the reinforcement (unless placed by depressors or tubes) be prepared well ahead of the paver or paving train.

A variety of techniques has been used to construct CRCP particularly on projects where loose longitudinal bars have been used. When the steel reinforcement is laid out ahead of the paving operation on supporting chairs the same concreting equipment is used for CRCP as for JCP; either slip-form pavers or rail mounted trains can be employed with, of course, the omission of the jointing machines.

4. DESIGN OF CRCP

A number of different design methods or recommendations are in use but as most of these are of recent origin the efficacy of the designs is hard to judge. All the designs recommend a reduction in thickness for CRCP when compared with JCP but the extent of the reduction is not agreed by different authorities. It is clear that with the flow of data from existing CRCP roads the design methods will be further refined as has been the case with concrete roads constructed with joints.

4.1 Comparison of CRCP and JCP designs (See Appendix 1, 12.5)

Major concrete roads in Great Britain are designed in accordance with the requirements of Technical Memorandum H10/71 which are based on the recommendations of Road Note No 29. Any design method for CRCP for use in this country should at first be derived from these recommendations.

After examining the evidence available the Study Group have concluded that for roads carrying 11 msa (million standard axles) or greater a reduction of 30 mm could be made in CRCP roads associated with a steel percentage of 0.6.

5. ECONOMIC ASPECTS OF CRCP

5.1 Initial costs (See Appendix 1, 13.1)

It is concluded by the Study Group that the initial costs of CRCP based on the recommendations in Sections 2.2 and 2.4 will be greater than for JCP designed to Road Note No 29 recommendations. This is because the increased cost of the reinforcement outweighs the savings made in concrete, joints and sliding layer. For a very heavily-trafficked road, the increased cost is likely to be between £0.23 and £0.66 per m². The Road Note No 29 design for continuously reinforced base with a bituminous surfacing is, however, likely to be more expensive than CRCP.

5.2 Maintenance costs (See Appendix 1, 13.2)

One of the main reasons for the widespread usage of CRCP in both the USA and Belgium is said to be the significant reduction in maintenance costs for CRCP roads compared with roads of other types of construction. This reduction in maintenance costs is attributed to the absence of transverse joints. The principle of combining initial construction costs with those associated with the subsequent maintenance of a road pavement was considered in RRL Report LR 256. On the basis suggested by that report if there is no maintenance requirement for transverse joints then on some very heavily trafficked roads where any traffic delays due to maintenance operations will be considerable the costs of these delays are likely to make CRCP economically viable. On other roads the reduced maintenance costs, particularly when discounted, are unlikely to compensate for increased construction cost.
6. CONCLUSIONS

1. Continuously reinforced concrete pavement is a practicable form of construction which can be considered as an alternative to reinforced and unreinforced jointed concrete pavements. Subject to the normal limitations for cold weather working CRCP can be laid throughout the year without design changes.

2. The construction of CRCP is relatively simple and the learning period with this technique would be shorter than that experienced with other recent innovations to JCP.

3. No detailed and long-term comparative studies have been made of the performance of CRCP and JCP but on the evidence available the Study Group feel that some reduction of pavement thickness is permissible when CRCP is used. For roads designed to carry more than 11 msda during the design life the permitted reduction recommended is 30 mm for a minimum steel percentage of 0.6.

4. Users of CRCP in the USA are of the opinion that when CRCP is properly designed and constructed the maintenance is significantly reduced compared with JCP. Again there is a lack of long-term documented evidence, but the Study Group is of the opinion that there is a real saving in maintenance. Belgian experience is too recent for any conclusions to be drawn.

5. As an alternative, to be selected by current tendering procedure in the UK, CRCP is unlikely to be competitive. On heavily trafficked roads at current material prices it is likely to cost between £0.23 and £0.66 per m² more than the other concrete alternatives. However it could be cheaper than CRC base with bituminous surfacing as per Road Note No 29 design.

6. If maintenance of CRCP is negligible, CRCP may have particular uses in situations where traffic delays from maintenance operations are least acceptable.

7. Some evidence exists that CRCP performs satisfactorily where problems due to differential settlement may arise. CRCP could also have applications in areas liable to settlement due to mining subsidence.

8. The many fine cracks which occur in CRCP do not affect the ride which experience showed to be very good.

9. CRCP can be used to overlay any existing pavement.

7. RECOMMENDATIONS

Although CRCP is potentially a viable form of construction it seems unlikely that it will be an economic alternative to existing forms of pavement currently used in inter-urban roads if only initial construction costs are considered. However, in congested urban conditions where delays for maintenance operations are becoming increasingly unacceptable from both economic and social viewpoints there would appear to be a good case for using CRCP, albeit on a limited controlled basis initially, to gain experience with the form of construction.

With the increasing emphasis of work in urban areas the time would seem appropriate to gain first-hand experience with CRCP, if full advantage is to be taken as the urban road programme expands.
8. APPENDIX 1

INFORMATION OBTAINED FROM THE AMERICAN AND BELGIUM VISITS

8.1 Symbols and metrification

\( A_c \) = area of concrete

\( A_{sl} \) = area of longitudinal steel

\( A_{st} \) = area of transverse steel

\( b \) = distance between free longitudinal edges

\( E_b \) = bond modulus

\( E_c \) = modulus of elasticity of concrete

\( E_s \) = modulus of elasticity of steel

\( f \) = stress in steel

\( f_b \) = bond stress

\( f_{c \text{ lim}} \) = elastic limit of steel

\( f_s \) = allowable working stress in steel

\( f_t \) = tensile strength of concrete

\( l_{\text{act}} \) = active length (at end of slab)

\( l_b \) = active bond length

\( L_{cr} \) = crack spacing

\( m \) = modular ratio = \( E_s/E_c \)

\( p \) = area of longitudinal steel per unit area of concrete

\( = A_{sl}/A_c \)

\( p_s \) = percentage of steel = \( p \times 100\% \)

\( u \) = perimeter of bar per unit area of steel

\( w \) = crack width
\( x \) = movement at end of slab
\( a_c \) = coefficient of thermal expansion of concrete
\( a_s \) = coefficient of thermal expansion of steel
\( \gamma \) = weight of slab per unit area
\( \delta \) = coefficient of shrinkage of concrete
\( \mu \) = coefficient of friction between slab and sub-base
\( \Sigma u \) = circumference of bars
\( \Delta T \) = total temperature drop from temperature at time of construction
\( \phi \) = diameter of steel bars

*Metrication*

All the dimensions used in reference to American practice (ie Imperial Units) have been directly converted to SI metric units.
9. THEORETICAL CONSIDERATIONS OF CRCP

CRCP as a form of construction can be examined from a theoretical aspect by considering that the concrete and the steel act separately. The concrete, and in part the sub-base and subgrade, carries the traffic stresses; the main function of the steel is to hold the cracks in a tightly closed condition so ensuring load transfer across the cracks by aggregate interlock. In considering the separate modes of action of the concrete and the steel due account must be taken of the fact that many of the properties of the concrete are subject to change, particularly at early stages (eg strength, modulus of elasticity, thermal coefficient of expansion), while in comparison the properties of the steel are relatively unchanging.

It is proposed that the theory of CRCP be examined from four aspects, namely,

1) support conditions,
2) slab thickness and concrete strength,
3) reinforcement and
4) terminal effects.

9.1 Support conditions

On the assumption that the most suitable criterion of performance of a concrete road, whatever its type, is the deflection at the surface under a given load then the effect of subgrade and sub-base strength can be assessed by use of the Westergaard formulae. These show that the deflection is inversely proportional to the square root of the modulus of subgrade reaction (k). Thus stronger support conditions in depth will reduce the deflections. Abou-Ayyash and Hudson used a discrete-element slab model to analyse CRCP and observed a definite logarithmic linear trend of subgrade modulus with deflection.

A thin unbound sub-base will have only a small effect on slab deflection. Cemented sub-bases have a larger effect but other reasons than reduction of the slab deflection exist for justifying the use of sub-bases and these are outlined later.

9.2 Slab thickness and concrete strength

There are many theories which have been applied to the analysis of stresses and deflections which occur in concrete pavement slabs. In all these theories the position of the wheel load relative to the layout of the slab has a very important influence on both the stresses and deflections. Most theoretical designs are based on the stresses and deflections due to the application of wheel loads at the edge or corner of the slab. A comparison has been made using the Westergaard formulae, as modified by Teller and Sutherland, of the stresses at the bottom of the slab for different load positions, support conditions and tyre pressures. In Appendix 1 Figs 1 and 2 the relationships are shown of the thickness of slab for edge and corner loadings which will produce the same tensile stress under a given load as that load placed in an interior position. Appendix 1 Figs 1 and 2 show that possible reductions of slab thickness ranging from 25 to 40 per cent can be achieved if slabs are designed on an interior loading condition instead of edge or corner loading conditions. McCullough and Ledbetter found that a 20 per cent saving in thickness could be achieved in concrete thickness over conventional jointed pavements designed for corner loading.
However in a concrete road the bulk of the wheel loads are concentrated at least one metre from the edge of the pavement and edge and corner loading conditions do not strictly apply. Any reduction in thickness made for CRCP will be based on the superior load transfer across cracks held together by reinforcement as compared with dowelled joints in JCP.

The argument can be made that a CRC slab in which there are many closely spaced transverse cracks is not suitable for analysis by the Westergaard theory because of the small sections into which the slab forms. Zuk\(^7\) assumed that the slab is homogeneous and continuous only between cracks and there is only partial continuity at cracks. This method of analysis provides a rational basis for determining slab thickness but cannot be readily used until more is known about the forces of aggregate interlock and also of reduced slab rigidity.

### 9.3 Reinforcement

The amount of reinforcement in a CRC slab is usually referred to as being \( p_s \) per cent; the value \( p_s \) is the cross-sectional area of the longitudinal steel expressed as a percentage of the cross sectional area of the slab.

Longitudinal reinforcement serves a different purpose to that of the transverse reinforcement and each type will be considered separately.

#### 9.3.1 Longitudinal reinforcement

The design aims at providing an optimum amount of longitudinal steel of suitable type so that fine cracks occur in the slab at suitable spacings. If the spacing of the cracks is too wide the cracks themselves will become wide with a consequent loss in load-transfer by aggregate interlock and accelerated corrosion of the steel; if the crack spacing is too close disintegration of the slab will commence. The function of the longitudinal steel is to keep the cracks in the concrete tightly closed thereby ensuring load-transfer across the cracks and also preventing the ingress of water and grit into the cracks.

In 1933 Vetter\(^8\) proposed a basic relationship that the percentage of steel required to control volume change in reinforced concrete was equal to the tensile strength of the concrete divided by the tensile strength of the steel. This relationship was based on the stresses produced only by shrinkage and temperature changes, and to prevent yield occurring in the steel the percentage of steel \( (p_s) \) is given by the formulae:-

\[
p_s = \frac{f_t}{f_y} \times 100
\]

where \( f_t \) = tensile strength of concrete

\( f_y \) = yield strength of steel

This expression is for the minimum steel percentage required for shrinkage in normal cases; however, the steel requirements for the resistance of any shrinkage are given by:-

\[
p_s = \left( \frac{f_t}{f_y + \delta E_s - m f_t} \right) \times 100
\]

where \( \delta \) = expansion constant

\( E_s \) = modulus of elasticity of steel

\( m \) = modulus of elasticity of concrete

\( f_t \) = tensile strength of concrete

\( f_y \) = yield strength of steel
where $\delta$ = coefficient of shrinkage of concrete

$E_s$ = modulus of elasticity of steel

$m = \frac{E_s}{E_c}$

$E_c$ = modulus of elasticity of concrete

Volume changes of the concrete caused by temperature require a minimum amount of steel which can be expressed thus:

$$P_s = \left( \frac{f_t}{f_y - mf_t} \right) \times 100$$ .................................. (3)

or by the following formula, whichever gives the greater value of $P_s$:

$$P_s = \frac{f_t}{2(f_y - \Delta T \alpha_s E_s)} \times 100$$ .................................. (4)

where $\Delta T$ = total temperature drop from temperature at time of construction

$\alpha_s$ = thermal coefficient of expansion of steel

Equations (1) to (4) inclusive will give values of $P_s$ generally of the same order, with equation (3) being found to be the critical one except where there are large temperature swings when equation (4) will be the most important.

All of the above relationships show that $P_s$ is directly related to $f_t$: this is of great significance because when concrete with high strength is used a greater amount of steel is necessary for an adequate design than with concrete of a lower strength. It might be argued from this relationship that there is a case for an upper limit on strength being placed on the concrete as well as a lower one (which is from durability aspects) when a CRC design has been agreed; variation of the steel content in a slab is not practicable to suit variations in concrete strength which, in any case, are not usually known until at least 7 days after the concrete has been laid.

Equations (1) to (4) are correct if it is assumed that the coefficient of friction between the underside of the slab and the surface of the sub-base has a value of 1.5. If $\mu$ varies considerably from 1.5 (ie, the sub-base is either very smooth ($\mu = 1.0$) or is very rough ($\mu = 2.0$)) then equations (1) to (4) require modification by a factor of $(1.3 - 0.2\mu)$. The implications of this are that the amount of steel required is greater when the sub-base is smooth than when it is rough.

Vetter also developed a formula which enables the spacing of cracks to be calculated; this is:
Equation (5) shows that the crack spacing \( L_{cr} \) is inversely proportional to \( p, u \) and \( f_b \); consequently to obtain fine cracks at close spacings the reinforcement percentage should be great as also should be the perimeter of the bars compared with their area. A close spacing of cracks is also obtained when the bond stresses are high and for this reason the use of deformed bars is to be preferred.

Another theoretical approach to the problem of crack formation and spacing is that suggested by Friberg\(^9\) which has been mentioned by Martin\(^10\) as adequately explaining the cracking in CRCP which has been observed in service. This approach considers the central position of the long slab in which 'active' cracks ie, cracks which are full-depth and which open and close when temperature changes occur, have formed. When the temperature drops below that at the time of construction the cracks open and slip occurs between the steel and the concrete close to the crack. The length over which this slip occurs is defined as the active bond length \( l_b \) and this may be calculated as follows:

\[
l_b = \frac{f}{E_b}
\]

where \( f \) = stress in steel at crack

\[
E_b = \text{bond modulus which is the decrease in steel stress for each unit length of active bond}
\]

\[
E_b = \frac{4f_b}{\phi}
\]

\[
f_b = \text{bond stress over length } l_b
\]

\[
\phi = \text{diameter of steel bar}
\]

The major factor determining the spacing of 'active cracks' is the initial slab temperature drop. This crack spacing can be determined from the following formula (assuming \( a_s = a_c \))

\[
L_{cr} = \frac{f^2}{E_b(E_s a_s \Delta T - mp(f - E_s a_s \Delta T))}
\]
The width of cracks is calculated from the following formula:

\[ w = \frac{f^2}{E_s E_b} \]  

Equations (7) and (8) both show that the crack width and the spacing of the cracks are inversely proportional to the bond modulus which itself is inversely proportional to the bar diameter. These formulae suggest that the spacing and widths of cracks are directly proportional to the diameter of the bar; this is in agreement with the conclusion drawn from equation (5).

9.3.2 Transverse reinforcement  It is usually considered that the main purpose of transverse reinforcement is to control any longitudinal cracks which may occur in slabs of normal widths, i.e., up to 4.6 m, and to keep such cracks tightly closed. Transverse reinforcement also serves to maintain the required spacing of the longitudinal steel and in the case of pre-set reinforcement it aids in supporting the longitudinal steel; it may also be used in lieu of tie-bars across longitudinal joints. In a well compacted slab with concrete of adequate strength longitudinal cracks are not likely to occur and therefore transverse reinforcement is not necessary from this aspect.

Zuk extended Vetter’s analysis to plain welded wire mesh in which the transverse bars provide an anchorage for the steel within the concrete. This study shows that the crack spacing and the crack width may be directly controlled by adjustment of the distance between the transverse wires; closer spacing of these wires means closer crack spacing and hence small crack openings.

When transverse steel is used the amount required can be calculated from the ‘subgrade drag’ formula (used in the USA for determining the reinforcement requirements for conventional JCP) which is:

\[ A_{st} = \frac{\mu b \tau}{2f_s} \]  

where \( A_{st} \) = cross-sectional area of transverse steel per foot width (in²)
\( \mu \) = friction factor (between slab and sub-base)
\( b \) = distance between free longitudinal edges
\( \tau \) = weight of pavement slab
\( f_s \) = allowable working stress in steel

9.4 Terminal effects

A long slab of CRCP subjected to changes in temperature will exhibit changes in length because of the temperature changes. The central portion of the slab will not move but lengths at either end of the pavement will move and the amount of movement will depend on the friction between the slab and the sub-base. By assuming the coefficient of friction to be constant the ‘active’ length can be calculated:

\[ l_{act} = \frac{\Delta T \varepsilon c}{\mu \tau} \]  

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where $l_{act} =$ active length

$a_c =$ thermal coefficient of expansion of concrete

If $x =$ movement of one end of the slab (half amplitude of total movement) this can be expressed as:

$$x = \int a_c \Delta T \, dl - \int \frac{\mu \tau} {E_c} \, dl$$

(11)

By integrating and substituting for $l$ it may be shown that

$$x = \frac{(a_c \Delta T)^2 E_c} {2 \mu \tau}$$

(12)

This demonstrates that for given temperature conditions the end movement is directly proportional to the elasticity of the concrete and inversely proportional to the friction factor. This again supports the theoretical need for a rougher sub-base which reduces the end movement (and also the longitudinal reinforcement requirements).

10. BEHAVIOUR OF CRCP

10.1 American experience

10.1.1 Effect of support conditions  Because CRCP is more flexible than JCP and the comparatively small effect of a thin sub-base the original thought was that the subgrade would not be overstressed even if the sub-base was omitted. The experimental pavement at Stilesville, Indiana 11 was 178 mm thick (with edges thickened to 229 mm) and this was laid directly on a poor subgrade prone to pumping. The sections with a high percentage of steel (1 per cent) have given excellent service since the road was built in 1938 including severe wartime usage by heavy military vehicles. However, a proportion of the failures in some of the early CRCP roads has been attributed to inadequate drainage and to either the omission of a sub-base or to an inadequate sub-base eg in Ohio 12 the use of an unstable granular sub-base is assessed as a primary factor in a number of failures.

In all the States visited, emphasis was placed on the necessity to prepare carefully (eg lime stabilisation in Mississippi) and adequately drain the subgrade. In addition the current trend is to place a strong sub-base beneath the pavement.

Some examples of current designs for sub-bases are; Texas uses the same sub-base as that required for JCP and particularly in the Houston area a 152 mm thick sub-base of sand-oyster shell stabilised with either cement or asphalt is used. Louisiana now uses a 152 mm thick cement treated sub-base (CTS) and in Mississippi the sub-base varies with soil conditions between 152 mm soil-cement base and 152 mm lime treated sub-base or 102 mm asphalt concrete base and 152 mm granular sub-base or 152 mm lime treated sub-base.
10.1.2 Effect of slab thickness and concrete strength

10.1.2.1 Slab thickness The recommendations for slab thickness suggested by the Continuously Reinforced Pavement Group (CRPG)\(^{13}\) and widely used by many States are that the thickness of CRCP should be between 70 and 80 per cent of the thickness required for conventional jointed concrete pavement. This is based upon empirical data and an analysis of stresses at cracks and joints. In support of these recommendations are quoted the results obtained in Texas\(^{14}\) where many measurements of deflection, radii of curvature, crack widths and temperature have been made on numerous non-experimental CRC and JCP roads. The most frequently quoted result from this research is that for a given wheel load 203 mm thick CRC pavements deflect about the same amount as 254 mm thick jointed pavements. As great importance is placed on this conclusion by the proponents of CRCP this data requires further examination. These results were based on a small sample, particularly of the jointed pavements, of normal in-service roads in which there are the usual variations of thickness, strength and support conditions. The results quoted do indeed show that a 254 mm thick JCP slab deflects at the joint 1.6 times as much as a 203 mm thick CRC slab deflects at a crack; at a midspan position (midway between two cracks) the 254 mm JCP slab deflects 1.4 times as much as the 203 mm CRC slab. However, the results are also given for a comparison between 203 mm CRC and 229 mm JCP slabs; here for the crack and joint deflection comparison the 225 mm JCP slab deflects only 1.15 times as much as the 203 mm CRC slab, and for the midspan position the deflection of the JCP slab is only 0.8 times that of the CRC slab. These anomalies are due to the small number of samples of highly variable factors and hence too much reliance should not be placed on these results.

This 80 per cent rule (or 203 mm CRCP = 254 mm JCP) has been widely accepted in the USA and most States use 203 mm thick CRCP for all major routes except the very heavily trafficked roads in or around major conurbations. The general claim (though this is not substantiated) is that 203 mm thick CRCP has a longer life with appreciably less maintenance than 254 mm JCP. The only State to query the adequacy of slab thickness is Ohio. In this State’s report\(^{12}\) the conclusion was that pavement failures have been confined primarily to the outside (nearside) lanes which carry the bulk of the heavier traffic loads, and the validity of present design procedures is questioned.

During the discussions with engineers at the Portland Cement Association the suggestion was made that there was no evidence to support the 80 per cent rule nor was there evidence to support the claims of longer life for CRCP. This view was also made during talks at the Federal Highway Administration Research Station.

10.1.2.2 Concrete strength and quality Minimum requirements for concrete strength vary from State to State but generally a minimum 28 day cylinder crushing strength of between 20.7 and 27.6 MN/m\(^2\) is required and average cube strengths are in all probability similar to those used in Great Britain. A typical concrete mix is that used in Texas for all major roadworks, ie 306 kg of cement per m\(^3\). Air entrained concrete is used in many but not all States. The quality of concrete is not generally considered to be a major factor affecting the performance of the road (CRCP or JCP) except that in CRCP faults during construction quickly show as failures, eg in an area containing a batch of undersanded poorly compacted concrete distress will soon be evident.

The quality of concrete is generally not high on the list of reasons for failures but a number of State highway engineers (in Texas, Mississippi and Washington, DC) expressed some disquiet at the degree of compaction of concrete laid by slip-form pavers travelling at speeds of about 6.1 m/minute. Despite this disquiet, physical testing of the concrete seemed to be spasmodic. The system appeared to be to agree the
mix proportions and workability at the start of the work, and thereafter to assume that all was well. This was surprising as usually a defects-repair period is not included in most contracts.

10.1.3 Effect of reinforcement The generally accepted quantity of steel is 0.6 per cent and formulae for the calculation of this percentage have been discussed in Section 9.3.1 of this appendix. Treybig\textsuperscript{15} in the final report of the performance of CRCP in Texas has concluded that 0.5 per cent reinforcement is sufficient to control the crack width and that percentages beyond 0.6 will not decrease deflections significantly. Abou Ayyash of the University of Texas states that there is no advantage to be gained by the use of percentages of reinforcement in excess of 1.0. Most American highway engineers believe that for their concrete strengths 0.6 per cent reinforcement will control crack widths such that there will be adequate aggregate interlock at the crack faces and experience shows that excessive crack widths are invariably associated with faulty construction techniques.

In most of the pavements inspected the reinforcement is placed between mid-depth and 64 mm from the top surface; this positioning is widely accepted. High bond deformed bar reinforcement is now invariably used either in the form of bar mats or of fabric; most engineers reject smooth round bar or drawn wire reinforcement on the basis of its unsatisfactory performance, ie large crack spacing and cracks of excessive width.

None of the American engineers considered that corrosion of the steel was a problem and only minimal evidence of rust was observed on one road, the Stevenson Expressway in Chicago.

10.1.3.1 Crack spacing and width In the final Texas report\textsuperscript{15} the optimum crack spacing is assessed as 1.52 to 2.44 m and the distribution of crack spacing is considered as a good indicator of performance, ie a normal distribution will indicate a good performance, whereas a skew distribution is indicative of the likelihood of a poor performance. (This distribution is over the central portion of the CRCP slab; at the ends of a CRCP slab the spacing of the cracks is greater). In practice most States appeared to accept a closer spacing of cracks than the optimum, eg in Mississippi the crack spacing settled down at a spacing of 0.92 to 1.52 m after a few years;\textsuperscript{16} on the heavily trafficked Stevenson Expressway in Chicago the crack spacing is in the range of 0.30 to 1.83 m with an average of 1.07 m. There is little information available on crack widths although the Texas report\textsuperscript{15} recommends that a design should aim at a crack width of 0.25 mm.

In Mississippi\textsuperscript{16} on two experimental projects crack widths have been continuously recorded. These results show the mean crack width on completion of the road (March 1961) as 0.23 mm, two years later (January 1963) as 0.51 mm and eight years later. (January 1969) as 1.14 mm. The widths quoted are those measured at the surface and are, therefore, inclusive of any spalling of the crack edges. The performance of these Mississippi projects is considered satisfactory by that authority.

Lack of factual knowledge on crack widths and acceptable values for these widths is admitted in the USA and a programme of research into this aspect of CRCP has recently been started.

10.1.4 Effect of traffic The impression gained during the visit to the States was that traffic on the Interstate highways in between cities was light but that traffic in and around the major cities, eg Houston, New Orleans and Washington, DC, was heavy. The roads in the Chicago area were very heavily trafficked, almost to the UK's M1 standard and there was a high proportion of truck traffic.
Only a limited amount of information was available from the State engineers concerning traffic intensities. On a typical rural Interstate road, the I 55 north of Jackson, Miss., the cumulative total of 8,200 kg axles over the seven year period following construction was $2.55 \times 10^6$ (thought to be one direction). For a rural motorway in the UK the designer would allow for $8.60 \times 10^6$ 8,200 kg axles in the same period.

On the Stevenson Expressway in Chicago figures from the Illinois Department of Transportation show that the near-side lane carried $11.43 \times 10^6$ 8,200 kg axles in 7½ years up to May 1972. The average daily number of commercial vehicles using this dual three-lane road in the first year was 5,000 increasing to 8,450 in 1971. This compares with the very heavily trafficked southern end of the M1. On inspection it was seen that the cracks were closely spaced (average 1.07 m) and that at the surface the cracks were wide, but the pavement was said to be performing satisfactorily.

In the Ohio performance study of CRCP\(^1\)\(^2\) the conclusion is that traffic loading is a major parameter, particularly where there is a large proportion of heavy trucks. The authors state that the failures occur primarily in the nearside lanes of heavily trafficked roads and the adequacy of the design pavement thickness is questioned. A similar doubt was expressed by the engineer for the Houston area of Texas; for the next section of Houston's loop road he proposes thickening the slab from 203 mm to 229 mm to extend the life of the road because of the heavy volume of traffic.

10.1.5 **Terminal conditions** The most popular form of end treatment in current usage is the wide flange beam joint (sometimes referred to as the Burdell joint); this is often associated with conventional jointed slabs with dowelled expansion joints; a section of such a joint is shown in Appendix 1 Fig 3. Some States, however, still use an arrangement of lug type terminal anchors and a typical layout of this arrangement is shown in Appendix 1 Fig 4. Wide rubber 'bellows' types of joint have not proved successful in service and are costly to instal and to maintain.

10.1.6 **Maintenance** All the highway engineers met on the tour stressed the benefit of the reduced maintenance associated with CRCP. The reduction in maintenance was always relative to the necessary maintenance of joints in JCP – generally jointed reinforced concrete. Clearly many States had experienced a lot of trouble with stepping and pumping at the transverse joints of JCP and the repair of faulty joints has given the highway engineers technical and political problems, particularly with the heavily trafficked roads in and around the major cities.

Generally pumping has been eliminated with CRCP and only Ohio reported a limited amount of edge pumping. The reasons for failure have mostly been attributed to either inadequate sub-base, areas of poor concrete, either through batching faults or because of lack of compaction, or insufficient overlap of the steel. The failures show as sections containing wide, closely spaced (less than 0.3 m apart), irregular cracks or as bifurcated cracks.

10.1.7 **Surface regularity** It was stated that the riding quality for CRCP was generally superior to that for JCP. A fair qualitative comparison between American and UK concrete roads in terms of the quality of the ride is impossible. Many American roads appear to give a much smoother ride than we are used to in the UK but the difference in ride is generally a function of the softer suspension characteristics of American cars and the absence of surface texture on their roads; the standard surface finish in the States is the burlap drag.
10.1.8 Use of CRCP in special ground conditions  There seems to be some evidence that CRCP performs well on poor subgrades, or where there is subsidence. The US 40 road at Stilesville, a CRCP road constructed in 1938 on a poor subgrade has performed well and sections are still in use. During the war this road carried a maximum of 4,300 vehicles per day of which approximately 60 per cent were trucks. The I 55 in the south of Mississippi has an undulating vertical profile due to cut and fill areas in an expansive clay but the pavement is in good condition and there is no sign of distress at the cracks. This better performance of CRCP is thought to be due to the increased flexibility of the pavement structure and hence its use in areas of subsidence should be comparable with the use of flexible construction.

10.1.9 General comments on American experience  The general conclusion of the State highway engineers met during the visit was that CRCP had performed well and that this form of construction met their requirements. It should, however, be borne in mind that the American system of awarding tenders is different from that obtaining in this country. The form of construction is usually laid down by the engineer who sometimes has to take into consideration factors other than engineering.

All the engineers stressed that the reduction in maintenance when compared with JCP was a very important item; however, it has not yet been possible to obtain any figures which substantiate this claim (American knowledge of the costs of maintenance is even less than British engineers' knowledge).

10.2 Belgian experience

In Belgium the first experimental length of CRCP was laid in 1950 and several other experimental lengths were constructed in the period 1958–1969. A party of Belgian engineers visited the USA in 1968 and since then CRCP has been an accepted form of construction in Belgium. As in the USA the form of construction to be used for new road construction is decided by the Administration and not by the contractors' as in Great Britain.

10.2.1 Support conditions  It is customary in Belgium to provide sufficient total thickness of construction to prevent damage of the subgrade due to frost action. Consequently most of the experimental lengths have been laid on substantial sub-bases; the only exceptions being two military roads laid directly on a sandy subgrade. Both of these roads had a low steel content (0.34 per cent) and the cracks were not water-tight. Current practice is to use the same thickness of lean concrete sub-base (200 mm thick) overlaid with a bituminous layer (60 mm thick) for both CRCP and JCP (unreinforced); in the JCP design the lean concrete sub-base is reinforced with square-mesh steel weighing about 4 kg/m².

10.2.2 Effect of slab thickness and concrete strength  As in the USA there has been no direct comparison of the effects on structural behaviour of either slab thickness or concrete strength.

10.2.2.1 Slab thickness  Thicknesses of 180, 200 and 230 mm have been used in different Belgian experiments. The first length laid at Leuze in 1950 is 180 mm thick and although the percentages of steel in this road are low (0.3 and 0.5 per cent) the road is still carrying heavy traffic (see 10.2.3). The current design recommendations for thickness are; for autoroutes and other heavily trafficked roads 200 mm, and for other roads with only light traffic 180 mm.

10.2.2.2 Concrete strength and quality  The type of concrete used in Belgium differs widely from that used in the USA and in Great Britain. A high strength and density are characteristic of Belgian pavement concrete; practically all the concretes are made with a porphyry aggregate (limestone has been used in two
of the experimental lengths). The concrete is not air-entrained but may contain a plasticiser (the only experience with air-entrained concrete was not successful). The specified minimum cement content is 350 kg/m$^3$; there is no precise strength requirement although cores cut from the road are tested and the strength of these cores at 56 days after suitable corrections are used in calculating payments to contractors. The level of the strength results is much higher than in the USA or in Great Britain with tensile strengths of the order of 3.4 MN/m$^2$.

The quality of concrete in Belgium is high and great care is taken to ensure full compaction. There is a stringent requirement for minimum density which if not attained involves removal of the concrete.

10.2.3 Effect of reinforcement All the Belgian work has been with deformed high strength steel bars, both in the form of prefabricated meshes and of loose bars. The early experiments had low percentages of steel (0.3 to 0.5 per cent) which have resulted in comparatively large crack spacings and wide cracks. At Leuze the section with 0.3 per cent steel had a crack spacing of 11.5 m after 18 years ie several times that regarded as desirable in the USA; when visited in 1972 it was evident that at many of these wide cracks the reinforcement had failed and faulting had occurred. The length with 0.5 per cent steel had a crack spacing of 3.5 m after 18 years; the cracks were wide (but not as wide as in the other section) and faulting had occurred at some of the cracks (Plates 3 and 4). The experimental road at Frasnes-lez-Gosselies (which is an 180 mm overlay of an old concrete road) contains 0.7 per cent, steel and the average crack spacing in 1967 after 3 years was 0.5 m. The percentage of steel currently used in Belgium is 0.85.

10.2.4 Terminal conditions The first three experiments had free ends and no provisions were made to restrict the movement at these ends. At Frasnes-lez-Gosselies one end of the section had an anchorage abutment in the soil while at the other end a 500 mm wide rubber joint was installed. Experience with this rubber joint has not been good and the practice at present is to provide abutments at the ends of CRCP sections.

Where it is possible the pavement is continued across bridges as shown in Appendix 1 Figs 5 and 6. If this is not possible an anchorage is provided for the pavement and between this abutment and that of the bridge two reinforced 10 m long slabs are placed which have dowelled expansion joints. This arrangement is shown in Appendix 1 Fig 7.

10.2.5 Maintenance Since the major programme of constructing CRCP is only about three years old there has been virtually no maintenance requirement but the Belgian engineers seem confident from their experience with the experimental roads that maintenance requirements of CRCP will be negligible.

The failure due to inadequate steel content of the road at Leuze has already been mentioned. Some minor failures in the road at Frasnes-lez-Gosselies were noted during the visit; these had been attributed to insufficient overlap of the longitudinal steel and had manifested themselves by very closely spaced transverse cracks.

10.2.6 Surface regularity Requirements for regularity on Belgian roads are strict and the contractor is liable to deductions for non-compliance (bonuses are paid for work better than that specified). The ride over new CRCP pavements and also those that had been carrying traffic for about three years was assessed as very good; some of the newer JCPs gave a good ride but there were some inconsistencies on this type of road (particularly at end of day joints) and the general opinion of the Study Group members was that the CRCP gave a better riding quality. This opinion was based on rides in Continental cars and on textured surfaces fairly similar to those now used in Great Britain.
10.2.7 General comments on Belgian experience  CRCP is now very much an accepted form of normal construction for autoroutes and other heavily trafficked roads in Belgium. The experimental programme is being continued both on existing lengths and on new work; a length was being laid in late 1972 in which a higher strength steel will be used. The concrete used throughout Belgium is very similar in all parts of the country and therefore standard designs are to be expected.

10.3 Overlays

CRC may be used to overlay an existing road which requires strengthening; the old road can be of either flexible or concrete construction. Some lengths of CRCP overlay were seen on I 69 in Indianapolis, one length with 0.6 per cent steel and one with 0.9 per cent steel were laid on a polythene bond breaking layer and the other with 0.6 per cent steel was laid directly on the old slab. It was however, too early in the life of these sections to determine the best section. Texas is proposing to use a tapered section overlay on one road ranging from 203 to 305 mm thick. Martin recommends that for very heavy traffic a thickness of 152 mm CRCP overlay can be used. He also recommends that partially bonded overlays should not be used over existing pavements having joint spacings greater than 18.3 m because of the large movements which may occur at these joints.

In Belgium the road at Frasnes-lez-Gosselies is a 180 mm thick CRC overlay laid directly on to an old 200 mm thick unreinforced concrete road (with 200 mm thick lean concrete for widening of the pavement). Another overlay in CRC seen in Belgium was the road between Barry and Bury; the old pavement, the construction of which is not known but which is thought to be flexible was treated with a levelling course and a 200 mm thick slab was laid on this.

The use of CRC for overlays is likely to increase in both the USA and in Belgium as more of the roads built just after the war to lower standards and for lighter traffic reach the end of their useful life as a surfacing.

11. CONSTRUCTION OF CRCP

11.1 American practice

11.1.1 New construction  Much of the work in the United States has been carried out at high rates of laying, sometimes reaching 6.1 m a minute. The same paving machines have been used, both slip-form and rail mounted, as for other types of concrete construction except, of course, that no jointing machines are necessary. Most of the work in America has been carried out with the concrete placed full depth in one operation. This requires either that the steel reinforcement is placed ahead of the paver on supporting chairs or fed into the concrete at the required depth.

All the steel is in the form of deformed bars. These may be factory welded into mats or alternatively may be delivered to site as loose bars. When the latter are to be used as pre-positioned reinforcement one procedure adopted is to set up the transverse bars on chairs and to locate the longitudinal bars on top of the transverse bars. The transverse bars are all slightly longer than half the pavement width so that by staggering their position the excess lengths may be used to provide a tie across the longitudinal joint. Cranes are sometimes used to speed up handling of the bars which are carried in specially devised slings. Because of the heavy steel used, it is generally necessary to handle the fabric reinforcement also by cranes.
For the steel which is to be depressed through the concrete either loose bars or fabric may be used. One method of laying the loose bars involves the use of a notched wheel which carries and places the transverse bars at the proper longitudinal intervals. The longitudinal bars are fed over the top and then brought down through guides to give them their correct transverse location, the whole of the steel then being vibrated to its correct depth. The transverse bars have been omitted altogether in certain contracts. Where fabric reinforcement has been used for paving between fixed forms the concrete has first been spread to full depth and struck off and then the fabric has been dispensed from a carriage following the spreader. Next, a special machine has depressed the fabric to its correct depth within the slab, followed by the compacting and finishing machine.

Pre-positioned steel reinforcement is satisfactory where the sub-base levels are sufficiently well regulated. Depressing the reinforcement through the slab has advantages where the sub-base regulation is poor or for the overslabbing of badly deteriorated pavements.

General recommendations as to the spacing of the longitudinal bars as given by the Continuously Reinforced Concrete Pavement Group in the United States are that the bars should not be closer than 102 mm to allow the concrete to pass freely between the bars and not more than 229 mm for good load transfer and adequate bond strength, larger diameter bars being normally associated with the larger spacing. The spacing of the transverse wires is usually determined from a formula involving the slab thickness, its width, the value of subgrade restraint and the working stress of the steel. Application of this formula results in spacing the transverse bars from 0.56 to 1.22 m.

Opinions differ as to the optimum depth of the steel within the slab. For instance in Texas it is recommended that the steel be placed at mid depth since this results in less total vertical movement under wheel loads and less stress in steel at cracks due to temperature differentials and wheel loadings. On the other hand it has been observed from cores cut in CRC pavements that cracks generally taper in width from the top of the slab to the level of the steel and remain fairly uniform below that level and this would suggest that the steel should not be placed too deeply. Generally the longitudinal steel is required to have a top cover of at least 64 mm.

The Continuously Reinforced Pavement Group also recommend that the tolerances on positioning of the longitudinal and transverse wires should be respectively ±12 mm and ±51 mm. The usual tolerance on the depth of the steel is ±6 mm. Almost always the requirement is that the longitudinal steel be placed above the transverse steel, and the reason generally given for this is that in this situation the transverse steel is less likely to generate or influence transverse cracks.

With CRC pavements the overlapping of the longitudinal steel is of special importance as experience has shown that where this is inadequate or faulty the result is often a localised failure in the finished slab. The invariable practice in the United States is to skew or stagger the overlaps in the longitudinal bars. Where fabric is used only one third of the overlaps are permitted at any one cross section. Where separate longitudinal bars are used and set up on the site the more general practice is to skew the line of overlaps at an angle of about 30° to the transverse.

The Continuously Reinforced Pavement Group recommends that the minimum overlap should be 25 diameters or 406 mm whichever is the greater. Specifications tend to err on the conservative side of this recommendation. For instance, in Mississippi the practice is to use overlaps of 508 mm. Welding of the overlaps is not permitted and they have to be wired closely together.
The end-of-day or construction joints are again areas of potential weakness and almost invariably specifications require special precautions to be taken in their construction. The usual specification is that all laps which occur in the last 0.92 m before the construction joint or within 2.44 m beyond it shall be twice the normal length, or if this is not practicable they have to be strengthened by splicing in an extra 1.83 m length of deformed bar of the same diameter at each overlap. There is normally no aggregate interlock at construction joints and therefore the practice is to instal across the joint at fairly uniform spacing a sufficient number of deformed bars of about 0.92 m in length to increase the area of the steel by at least one third.

At the location of the joint the reinforcement is carried through a split header. The practice is to support this steel very carefully to avoid its deflection and hence the possibility of disturbance of the newly laid concrete and to cover the steel with boards while work is carried out at the joint. Emphasis is placed on the need for careful compaction near the joint because of the congestion of steel.

11.1.2 Specification and tendering practices. Although specifications have the usual restrictions requiring the cessation of work at temperatures near freezing, no reference was found to an upper limit of temperature for concrete laying and paving sometimes continues when the ambient temperature is 32.2°C or more.

Looking at the general picture it is evident that in the United States the contractors' approach to CRC paving is influenced by the considerable volume of work available which gives him an assurance of continuity, by the relationships between labour, plant and transport costs and by the greater widths of construction which make for easier site movement than is the case in the United Kingdom and in certain other European countries. On Interstate schemes several earthworks and structures contracts are often grouped together for one large paving contract. In Mississippi the Study Group visited a site on which the sub-base preparation had been carried out throughout the whole length during the previous season by another contractor, the paving contractor being presented with a long stretch of good working surface.

Generally, specifications do not place emphasis on methods. In this context the contractor is encouraged to innovate, with the result that there are many variations in the method of construction, particularly with regard to the steel placing, some of which involve relatively expensive pieces of plant. The laying speeds, which are sometimes very high, involve considerable expenditure in batching and mixing plant and in transport for the concrete.

The choice of loose bars as against fabric reinforcement is influenced to some extent by geographical considerations as, for instance, in Maryland where various contractors engaged in construction of the I 95 from the Capitol Beltway to Baltimore all used loose bars. This was probably due to the proximity of a supplier of loose bars whereas the closest fabricator of mats was located in Pennsylvania.

11.1.3 Maintenance Maintenance of CRC paving in the United States is still to some extent experimental. For example, in both Texas and Mississippi the district offices have been given freedom in the manner in which repairs are carried out and observations are being kept on the relative success of the different techniques. Relatively few failures occur, but those which do are most often associated with faults in construction, such as inadequate overlaps in the longitudinal steel, poor compaction at the construction joints or honeycombing elsewhere under the reinforcement.

In one District in South Mississippi repairs were carried out where unacceptably extra wide cracks had appeared at the laps of fabric reinforcement. These repairs have so far proved successful and the technique is...
worth quoting. Firstly, two saw cuts were made 0.76 m to 1.27 m apart with the wide crack in the centre. These cuts were made 25 mm deep, after which a jack hammer was used to remove the concrete down to the reinforcement. After the concrete had been removed to the depth of the reinforcement and the area brushed clean, 12 mm reinforcing bars, 610 mm long, were placed and welded to the mats. Following the welding, the entire area, concrete and steel, was coated with epoxy glue. The concrete used to fill the void above reinforcement was made with high early strength cement and was vibrated into position. Curing was effected with wet burlap. Ten such repairs carried out in 1962 were reported as still sound in 1969. Obviously such a repair technique as that described could only be applied where the concrete at the bottom of the slab remained relatively sound.

The recommendations of the Continuously Reinforced Pavement Group for patching are as follows. The minimum length of patch should be 3.05 m and the repair should be placed on a 1 to 4 skew across the pavement to avoid both wheels of an axle crossing the construction joint simultaneously. A groove is then sawn about 25 mm deep at each boundary of the patch. Two further pairs of cuts, each pair about 152 mm apart, are then sawn parallel to and 0.92 m inside each of the boundary grooves. The concrete is then chipped out from between each pair of cuts as far down as the reinforcement, which is then cut with a torch or bolt cutter. All the concrete between the boundary cuts is then removed, leaving the reinforcing steel exposed at each side. A concrete breaker should not be used, except between the inner cuts where there is no fear of transmitting shock through the reinforcement back to sound concrete outside the patch. After any necessary restoration to the sub-base, deformed bars or deformed wire fabric are placed across the patch and spliced in to give an overlap of 0.92 m approximately, with the steel extending from both boundaries. The laps are wire tied but not welded. Supplementary bars or wires are also spliced in to increase the cross sectional area of the longitudinal steel by 50 per cent. These extra bars should run the full length of the patch. The reinforcement is all firmly supported at the proper elevation above the sub-base on steel supports.

If movement of the pavement ends causes the reinforcement to buckle, this should be corrected just prior to placing concrete by removing and replacing the wire ties at the laps. Because of the close spacing of the steel extreme care has to be taken in compacting concrete so as to avoid any honeycombing. The surface of the patch should be finished to provide a smooth ride, and the patch should be properly cured and not open to traffic until the concrete has attained a flexural strength of 4.1 MN/m².

There may be a good case for welding the lapped reinforcement at repairs, as done in Mississippi, rather than depending on wire ties as in these last recommendations, since it is very doubtful if wire ties could hold against the contraction of the neighbouring slab ends in the period before the concrete developed sufficient bond strength.

Engineers in Mississippi have found that concrete repairs completed in the afternoon are more likely to be successful than those completed in the morning and have suggested that the concrete should be placed when the neighbouring slabs are in their maximum state of expansion. Measurements made at one patch in Leflore County in Mississippi in 1968 showed that the slab ends either side of the patch closed by 27 mm between 8.30 in the morning and 4.20 in the afternoon, when further movement ceased.

11.2 Belgian practice

11.2.1 New construction Both rail-mounted and slip-form paving machines have been used to lay CRCP, but at outputs lower than in the USA and more like those achieved in this country. Laying rates of 215 m per day for rail-mounted machines set to construct paving 3 lanes wide have been typical, with a maximum
output of 300 m in one day. Corresponding figures for slip-formed paving 3 lanes wide are 500 m per day average and a maximum of 600 m.

The main features of construction are set out in a Belgian publication on continuously reinforced roads. The concrete is invariably spread and compacted to full depth in one layer with the reinforcing steel set out ahead supported on one of several types of steel chair or wire support (plates 5, 6 and 7). These have to be fastened to the transverse bars of the reinforcement, either by tying or welding. The recommended distribution of supports is three per square metre; they are staggered in the longitudinal direction, alternating between one pair of longitudinal bars and the next.

All the reinforcement is in the form of high yield strength (at least 490 MN/m²) deformed bars and may be either factory-made into fabric or made up on site into bar mats, although a preference for fabric was expressed because of the better geometric layout that it gave. The diameters of the longitudinal bars range from 14 mm to 20 mm and the transverse bars from 10 mm to 14 mm. The spacing between longitudinal bars ranges from 140 mm to 160 mm and between transverse bars ranges from 680 mm to 720 mm.

Longitudinal bars are invariably placed above the transverse bars and have to have a minimum cover of 70 mm measured from the axis of the bars. In arriving at the height of the supports necessary to achieve this, account has to be taken of the variability in the surface levels of the 60 mm bituminous layer which is always laid directly under the slab. Emphasis is placed on the need to achieve surface tolerances on the bituminous layer which are comparable with those on the finished slab and consequently this layer is usually constructed by plant electronically controlled off guide wires.

The transverse bars may be placed either perpendicular to the axis of the road or at an angle of 60°, whether fabric or bar-mat reinforcement is used. Precise arrangements for the transverse and longitudinal overlaps between mats or bars are specified. Curves of large radii encountered on the autoroutes or high speed roads do not cause any special difficulty in the layout of the steel.

The quantity of steel used, 0.85 per cent, is high in order to suit the high strength concrete. A plasticiser may be added to the mix, but because of the design strength, calling for a water-cement ratio of about 0.40, air-entraining agents are not considered necessary to achieve high durability. The publication places emphasis on the need for continuity of laying, high daily outputs in terms of length and for good quality control, requiring the slump to lie within a range of ± 15 mm. Across the end-of-day joints the longitudinal steel is doubled over a length of 1.50 m and the concrete is given extra compaction with poker vibrators.

11.2.2 Specification and tendering practice The construction of CRCP in Belgium is governed by the same specification as that for JCP with additional requirements for the steel. Tendering practice is similar to that of the USA and is usually arranged so that the paving contract is of a suitable length to allow the economic use of slip-form pavers.

11.2.3 Repair techniques The majority of the Belgian CRC pavements are comparatively new and there has been no need as yet for any repair work to be carried out on these roads. As was mentioned earlier there have been minor failures in some of the experimental roads but no special techniques have been evolved for repairs.
12. DESIGN METHODS FOR CRCP

For conventional concrete pavements, i.e. with transverse joints, there are many design methods available some of which are purely theoretical but the majority are of an empirical nature. Since the large scale use of CRCP is comparatively recent it is natural that methods of design for this type of construction should be based upon the accepted design methods for conventional concrete pavements with suitable modifications for the use of the continuous (and heavier) reinforcement. Ideally any method of pavement design should consider four broad classes of variables, namely load variables, structural variables, climatic variables and finally performance variables. It could also be argued that economic variables should be considered in any design assessment and this aspect will be discussed later in Section 13. Many of these variables cannot be accurately quantified, for instance, axle loadings and climatic conditions, and hence a compromise is introduced into the designs.

Consideration of the above mentioned design variables for any pavement type will lead to recommendations for the thicknesses and strengths of materials in the various layers of the structure; additionally for CRCP recommendations are required for the quantity and type of steel reinforcement, concrete/sub-base interface treatment and also for the design to be adopted at the ends of the pavement. Current designs procedures for CRCP will be discussed under the same headings as for the theoretical treatment in Section 9 of this Appendix. This will be followed by a comparison of CRCP designs for different support and traffic conditions obtained by using these design methods and also to compare these designs with those for conventional jointed concrete pavements and for continuously reinforced concrete bases with bituminous surfacings as recommended by Road Note No 29.1

12.1 Support conditions

It is now accepted in the USA and Belgium that a sub-base is desirable for a number of reasons. These are the prevention of pumping, the provision of a working surface for construction purposes and the reduction in stresses and deflections in the slab. There is, therefore, no reason for the support conditions beneath a CRC road to be different from those under a conventional concrete pavement.

12.2 Slab thickness and concrete strength

12.2.1 Slab thickness As has been mentioned in Section 9.2, theoretical methods of slab design derived for jointed concrete slabs can be modified so that they may be used for CRCP. None of these methods are in use for the derivation of slab thickness. As their validity has not been confirmed they will not be discussed further.

The recommendations for slab thickness suggested by the Continuously Reinforced Pavement Group\(^\text{13}\) (referred to in Section 10.1.2) are that the thickness of CRC should be 70 to 80 per cent of the thickness required for conventional JCP. In the AASHO interim method of design\(^\text{10}\) based on the AASHO Road Test results, a method of determining the thickness for JCP is detailed and the recommendation for CRCP is that the thickness of this form of construction should be 25 or 51 mm less than that of the JCP but no guide is given as to the conditions for the alternative deductions.

A recent design method is that proposed by the ACI\(^\text{20}\) this is based on the AASHO method with the inclusion of the elastic modulus of the concrete and of a factor J known as the load transfer factor. This factor may range from 1 to 5 depending on the form of construction. The concept of this factor J was first introduced in the analysis of the AASHO Road Test results and the value of 3.2 was derived for jointed concrete pavements with adequate load transfer at the joints. The ACI recommendations suggest that the
values of $J$ will be between 2 and 3 when adequate amounts of steel are used and until further information is available a value of $J$ of 2.2 is considered reasonable. The latter value is based upon an assumption made by Hudson and McCullough\textsuperscript{21} which fitted test results in Texas. In the ACI method the thickness is determined by the use of a nomograph (Appendix 1 Fig 8) in which it is necessary to know the cumulative traffic in equivalent 8,200 kg axles, the flexural strength and modulus of elasticity of the concrete, and the modulus of support.

A design method is not used in Belgium and the present recommendations (see Section 10.2.2.1) are based on the results of Belgian experimental CRC roads and on American usage suitably modified for the heavier wheel loads used in Belgium and also for the higher strength concrete which the Belgians use. If the American designs are accepted as valid then those in use in Belgium appear to be reasonable.

12.2.2 Concrete strength Theoretical considerations show that the deflections and stresses in a concrete slab are influenced by the strength of the concrete and this influence has been demonstrated in practice.\textsuperscript{22} In both the AASHO and ACI design methods referred to above the strength of the concrete (flexural) is used in determining the slab thickness. The Belgians have calculated the thickness requirement for higher strength concretes by equating moments of resistance for concretes of two different tensile strengths.\textsuperscript{23}

Section 9.3.1 has shown that the percentages of longitudinal steel is directly proportional to the strength of the concrete and therefore it must be borne in mind that with a higher strength concrete more steel is required.

Another factor which should be considered with regard to concrete strength is the rate of strength development; all the previous discussion has been related to concrete strengths developed at ages varying from 14 to 56 days. Experience has clearly shown that the crack pattern in a CRC slab is basically determined at a very early stage in the life of the pavement and therefore the rate of strength development during the early life of the slab is an important factor in determining the long term behaviour of the pavement. The only known experiment to determine the effect of cement type on performance was carried out in Texas\textsuperscript{24} where with a type III cement (high-early strength) a much greater average crack spacing resulted than in a pavement in which type I cement (normal) was used.

12.3 Reinforcement

12.3.1 Longitudinal reinforcement Equations (1) to (4) are used in the ACI recommendation for calculating the percentage of longitudinal steel. Equation (3) is normally found to give the highest value of $p_s$ and to calculate this equation, values for the yield strength of the steel (0.2% Proof stress), the tensile strength of the concrete and the moduli of elasticity of both steel and concrete are required. Values for all these factors are known or can easily be measured with the exception of the tensile strength of the concrete. As there are as yet no standard tests for measuring the tensile strength of concrete the ACI recommends that a value of $f_t$ equal to 0.4 times the 28 day modulus of rupture be used and that a safety factor of 1.3 be applied to the value of $p_s$ so obtained.

The Continuously Reinforced Pavement Group give no details of calculating the steel percentage but suggest that 0.6 per cent steel be used except in severe climatic conditions or for very heavy traffic when an increase in the amount of steel may be desirable. In Belgium\textsuperscript{17} the percentage of longitudinal steel is calculated from the formula:=-
where \( f_{e \lim} \) = elastic limit of the steel.

For the Belgian type of concrete the steel percentage is of the order of 0.85.

The manufacture of steel rods and bars to the length of the complete pavement is naturally impossible and to ensure continuity of the steel overlapping of the longitudinal steel is necessary. Tests have shown that with deformed bar reinforcement a 32-diameter overlap is adequate\(^{25}\) and the ACI recommendations suggest a minimum overlap of 30 diameters when the laps are in the same transverse section. When the laps are skewed the strain at the laps is less and the minimum lap may be reduced to 25-diameters provided not more than one-third of the bars are lapped within any 0.92 m length of pavement. In Belgium a skewed arrangement of overlaps is commonly used with the length of the lap being about 35 times the bar diameter.

Spacing of longitudinal steel should not exceed 203 mm according to the ACI recommendations and the CRPG suggest a maximum of 229 mm. The minimum spacing in American practice is 102 mm for single course work; where two course work is used (extremely rare) a minimum of 76 mm can be used.

### 12.3.2 Transverse reinforcement

The formula given in equation (9) is normally used in the USA to determine the amount of transverse steel. A limited investigation into the effect of transverse steel in CRC slabs has been made\(^{26}\) which concludes that the transverse steel ensures adequate load-transfer across any longitudinal cracks. This investigation also considers the probability of longitudinal cracks occurring and presents a cost-performance analysis based on this probability and the difference in initial costs. A maximum spacing of 1.22 m is suggested for transverse bars; with fabric the maximum spacing is limited by manufacturing processes to 406 mm.

In Belgium the transverse steel is specified and may be at either right angles to the longitudinal steel or at an angle of 60°; it is always placed beneath the longitudinal steel.

### 12.4 Terminal treatment

The movements at the ends of long CRC slabs due to temperature changes have been discussed in Section 9.4. As well as these seasonal movements there is ample evidence that pavement growth which is not reversible takes place as a result of creep. This can obviously have very undesirable effects at the ends of the pavement and at bridge abutments.

In designing terminal arrangements there are two options, firstly to allow free movement by means of a joint capable of absorbing all the growth, and secondly, to resist the movement either wholly or partially. The ACI recommendations suggest that expansion joints which can accommodate a minimum 51 mm movement be used, or that end anchorages with expansion joints capable of accommodating 25 mm movement be used. End anchorages are usually in the form of lugs cast in the subgrade and connected to the CRC slab by steel reinforcement; the lugs are generally transverse to the pavement but may also be longitudinal.

In Belgium wherever possible the pavement is made continuous across under-bridges, with the doubling of the reinforcement between points 10 m each side of the bridge, achieved by placing the extra layer of...
steel in the bottom third of the slab. Where the pavement cannot be taken across the bridge, lug type abutments, which depend on soil friction for their anchorage, are placed not closer than 30 m to the bridge to restrain movements in the slab-ends. Between these and the bridge abutments are constructed two reinforced slabs, not more than 10 m long, separated by dowelled expansion joints, followed by a short length of bituminous pavement. The expansion joints are to take up and absorb any residual movements at the ends of the slab, up to a design maximum of 5 mm, though such movements may not occur with well designed anchorage abutments. Similar anchorage arrangements are made at the ends of a length of CRC pavement.

12.5 Designs obtained with current recommendations

The design methods and recommendations in use at the present time have been arrived at as a result of theoretical and empirical studies and with the increasing use of CRCP in both the USA and Belgium it is worthwhile to compare the best known methods with the recommended designs for both normal JCP and for CRC bases with bituminous surfacings at present used in Great Britain (Road Note No 29). Designs will be prepared from the following: a) the ACI procedure,\textsuperscript{20} b) the CRPG recommendations\textsuperscript{13} c) the Belgian recommendations,\textsuperscript{17} d) Road Note No 29 design for conventional concrete pavement and e) Road Note No 29 design for CRC base with a bituminous surfacing. This comparison will show any differences in the structural designs of the different pavement and the economic implications of these differences will be discussed in detail in Section 13.

12.5.1 Design assumptions

Designs will be considered for four traffic loadings, ranging from light to very heavy traffic, and for three different soil conditions comparable to the classification of subgrades given in Road Note No 29. It is also proposed to make these designs for two different concrete strengths, the first approximating to the strength of typical British paving concrete, and the second having a strength similar to that of the concrete used in Belgium.

12.5.1.1 Traffic conditions

The traffic is expressed in terms of cumulative equivalent number of 8200 kg axles. The four traffic loadings selected are as follows:-

<table>
<thead>
<tr>
<th>Traffic Loading</th>
<th>Cumulative Number</th>
<th>Equivalent Traffic</th>
</tr>
</thead>
<tbody>
<tr>
<td>T1</td>
<td>50.0 × 10(^6)</td>
<td>8200 kg axles</td>
</tr>
<tr>
<td>T2</td>
<td>10.0 × 10(^6)</td>
<td></td>
</tr>
<tr>
<td>T3</td>
<td>1.0 × 10(^6)</td>
<td></td>
</tr>
<tr>
<td>T4</td>
<td>0.5 × 10(^6)</td>
<td></td>
</tr>
</tbody>
</table>

12.5.1.2 Support conditions

As the ACI method of design uses the modulus of support (k) in the nomograph an assumption has to be made of the values of the modulus of subgrade reaction and then the k value for sub-bases laid on these sub-grades. The three subgrades chosen are:-

<table>
<thead>
<tr>
<th>Subgrade</th>
<th>CBR</th>
<th>Modulus of Subgrade Reaction</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1</td>
<td>20%</td>
<td>6.76 MN/m²/m</td>
</tr>
<tr>
<td>S2</td>
<td>3%</td>
<td>2.71 MN/m²/m</td>
</tr>
<tr>
<td>S3</td>
<td>1.5%</td>
<td>1.35 MN/m²/m</td>
</tr>
</tbody>
</table>

These sub-grades are within the very stable, normal and weak classifications of Road Note No 29.
For the S2 and S3 subgrades it has been assumed that a 152 mm sub-base would be used; both granular and cement-treated sub-bases have been considered. Curves relating the modulus of support obtained by the use of varying thicknesses of sub-base on subgrades of varying strengths have been used to determine the k values on top of the sub-base which are:

\[
\begin{align*}
\text{S2g (Soil S2 with granular sub-base)} & : k = 3.52 \text{ MN/m}^2/\text{m} \\
\text{S3g (Soil S3 with granular sub-base)} & : k = 1.90 \text{ "}" \\
\text{S2c (Soil S2 with cement-treated sub-base)} & : k = 7.57 \text{ "}" \\
\text{S3c (Soil S3 with cement-treated sub-base)} & : k = 4.86 \text{ "}" 
\end{align*}
\]

### 12.5.1.3 Concrete strength

To use the ACI nomograph reproduced in Appendix 1 Fig 8 it is necessary to know the 28 day modulus of rupture and the modulus of elasticity of the concrete. It is not clear whether the modulus of rupture to be used in the calculations is the average value or the specified minimum value; for this comparison it has been assumed as the former. The modulus of elasticity of the concrete may be determined either statically or dynamically and it is well known that the values differ according to the method of measurement; since most of the cracking in a CRC slab is of a thermal nature it seems appropriate to use the static modulus. The two types of concrete examined are as follows:

\[
\begin{align*}
\text{C1} & : & \text{Modulus of rupture} & = 4.1 \text{ MN/m}^2 \\
& : & \text{Modulus of elasticity} & = 34.5 \text{ GN/m}^2 \\
\text{C2} & : & \text{Modulus of rupture} & = 6.2 \text{ MN/m}^2 \\
& : & \text{Modulus of elasticity} & = 37.9 \text{ GN/m}^2 
\end{align*}
\]

### 12.5.2 Comparison of thicknesses obtained from the different designs

In using the ACI nomograph, Appendix 1 Fig 8, extrapolation of Line I was necessary to obtain a design for the heaviest traffic considered; this was done logarithmically and an appropriate comment here is that the upper limit of 10 million 8,200 kg axle loads does not envisage the use of the chart for heavily trafficked roads such as motorways. Another comment on Appendix 1 Fig 8 which is relevant refers to the worked example; the use of compatible values of modulus of rupture and modulus of elasticity is emphasised but the value of $E_c$ appears rather low in relation to a MR value of 4.0 MN/m$^2$.

The thicknesses obtained for the CRPG method are the slab thicknesses obtained by using Road Note No 29 reduced by 25 per cent, ie between 70 and 80 per cent of the thicknesses required for normal jointed concrete pavement in Great Britain.

The slab thicknesses obtained by the different design methods are shown in Table 1 Appendix 1; no values are given for Belgian recommendations for use with the C1 type concrete since concrete of that strength is not used in that country. For the Road Note No 29 design for a CRC base with a bituminous surfacing no slab thicknesses are shown for traffic categories T3 and T4 as the recommendations for this type of construction only cover traffic loadings in excess of 2.5 million, 8,200 kg axle loadings. When considering the higher strength concrete (C2) the slab thicknesses for designs to Road Note No 29 have been obtained by the method suggested in LR 423; for the strengths of concrete used, the values of thicknesses for the C2 concrete are
0.8 times those required with the C1 concrete. All the derived thicknesses, ie for all except the Belgian recommendations, have been rounded up to the nearest 10 mm.

It has been stated earlier that the design of slab thickness and longitudinal reinforcement should be considered together and therefore conclusions drawn directly from Appendix 1 Table 1 must be considered in this context. However it is apparent that for the two heavier categories of traffic loading the ACI method gives slab thicknesses for CRCP which are thicker than the values obtained from Road Note No 29 for JCP. The effect of using a lower modulus of elasticity with a given modulus of rupture has been tested and the maximum reduction because of this is likely to be only 10 mm and in several cases there will be no change.

12.5.3 Comparison of reinforcement designs  The formulae for calculating the percentage of longitudinal steel have been given earlier in Section 9.3.1 (equations (1) to (4)). For comparative purposes only equation (3) has been used and the safety factor of 1.3 has been applied to the calculated value of $p_s$. The CRPG design manual\textsuperscript{13} does not give a method of design but reports the Federal requirement of at least 0.6 per cent and this value will be used as their recommendation.

In the calculations the following values have been assumed:-

\[
\begin{align*}
    f_t &= \text{tensile strength of concrete} = 0.4 \times \text{modulus of rupture} \\
    &= 1.7 \text{ MN/m}^2 \text{ for C1 concrete} \\
    &= 2.5 \text{ MN/m}^2 \text{ for C2 concrete} \\

    f_y &= \text{yield strength of steel} \\
    &= 412 \text{ MN/m}^2 \\

    E_s &= \text{modulus of elasticity of steel} \\
    &= 192 \text{ GN/m}^2 \\

    \text{Hence for type C1 concrete} \quad m &= \frac{E_s}{E_c} = 5.8 \\

    \text{and for type C2 concrete} \quad m &= 5.25
\end{align*}
\]

The derived values for $p_s$ are:-

for type C1 concrete 0.54 per cent

for type C2 concrete 0.81 per cent.

Simple comparison of steel percentages is not appropriate since the amount of steel in a particular design will depend on the slab thickness. From the above steel percentages and the slab thicknesses contained in Appendix 1 Table 1 the areas of longitudinal steel per metre width of slab have been calculated for the different designs and these are shown in Appendix 1 Table 2. Approximate weights per square metre are

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given in Appendix 1 Table 3. As for the slab thickness Table, no Belgian recommendations are shown for type C1 concrete and the amount of steel for the type C2 concrete is based on the 0.85 per cent suggested in the Belgian hand book.\(^{17}\) The CRPG recommendations only relate to typical American concrete and consequently no designs for steel percentages are given for higher strength concrete; the values for area of steel for the CRPG design thickness with type C2 concrete are based on a steel content of 0.85 per cent.

The steel areas per metre width for the Road Note No 29 jointed design are those obtained from Fig 12 of the Road Note and the values for the CRC base are those for the BS long mesh reinforcement for the recommended minimum weight of reinforcement.

In the calculation of the steel content for the ACI design the coefficient of friction (\(\mu\)) has been assumed to be 1.5; if the sub-base is very rough and \(\mu\) is taken as 2.0 then the steel requirement is reduced by 10 per cent and conversely if with a very smooth sub-base (\(\mu = 1.0\)) the amount of steel required is increased by 10 per cent.

12.6 Comments on the design methods

In this country experience with the CRC base and a bituminous surfacing has been very satisfactory. If CRCP were to be allowed for the running surface it would be essential to have design recommendations for this form of construction and the comparison which has been made allows the merits and disadvantages of the methods to be assessed. The question of economics is dealt with later but obviously the justification for slabs of only slight reductions in thickness from the JCP currently used with a considerable increase in the amount of steel will be difficult, if not impossible on the basis of initial cost.

The designs obtained by using the ACI method do not, in fact, offer any savings in slab thickness for the two heavier classes of traffic, indeed they indicate thicker slabs than those now used. This together with the very great increase in the weight of reinforcement would make these designs prohibitive. It is difficult to understand the reason for the thicker slabs and as the reductions in slab thickness are only appreciable for the lighter traffic on the poorer soils it can only be concluded that a higher standard is expected at the end of the design life than in the Road Note No 29 designs.

More attractive are the designs obtained by following the advice of the CRPG: this is to be expected since the thicknesses are based on a 25 per cent reduction in the Road Note No 29 designs and the steel percentage is then related to these reduced thicknesses. In considering the Belgian designs it is important to remember that the slab is laid on a 200 mm thick lean concrete on which is laid a 60 mm thick bituminous layer. For soil types S1 and S2 the Belgian designs appear very conservative.

British experience with continuous reinforcement is limited to the experiments at Grantham\(^{28}\) and Thurmanston\(^{29}\). In the former experiment it was concluded that at least 8.9 kg/m\(^2\) of reinforcement would be necessary for a 230 mm thick slab (0.46 per cent). In both of these projects there was no variation in slab thickness and therefore conclusions are not possible about the possible reduction in slab thickness as compensation for the increased steel content. Some light on the possible reduction is shown by the performance of conventional jointed reinforced slabs 36.6 m at Alconbury Hill.\(^{30}\) Here 178 and 203 mm thick slabs with reinforcement weighing 7.74 kg/m\(^2\) had after ten years performed as well as 229 mm slabs with 5.84 kg/m\(^2\); after 16 years the 203 mm slab has an equal performance with the 229 mm slab with the lighter reinforcement. This suggests to obtain the same performance a reduction in slab thickness can be made when the percentage of steel is increased.

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12.7 CRCP with 'elastic' joints

It has been shown that the width of cracks in CRCP is dependent on the spacing of cracks and if for any reason the spacing is wide then the cracks will also be wide. It is known that the spacing is greatly influenced by the climatic conditions at the time of construction along with the other factors. Since control of climatic conditions is clearly impossible it has been suggested that preformed cracks at predetermined intervals be induced in the slab; these cracks have been called 'elastic' joints.

Most work with this type of construction has been done in Sweden although other work has been done elsewhere on the Continent. With this type of 'elastic' joint it is possible to use a lower percentage of steel and also to use plain round wires. The Study Group have not been able to inspect any of this type of construction but is investigating this approach further with a view to its possible application in Great Britain.

13. ECONOMIC ASPECTS OF CRCP

So far roads with CRCP have been considered from the aspects of theory, design, behaviour and construction and it has been shown that this form of pavement is feasible from all these aspects. However, for CRCP to be viable it must compete on an economic basis with other forms of concrete roads (and flexible roads).

As no CRCP has been constructed in Great Britain as the running surface there are no factual data available on either initial construction costs or of the maintenance costs; any comparison of costs must be made on the basis of permissible design changes and on information available from the USA and from Belgium.

13.1 Initial costs

The differences in tendering procedure in both the USA and Belgium from those in use in Great Britain have already been mentioned. Because of these differences true cost comparisons between CRC and JCP laid in the two countries are difficult to obtain. In Mississippi the cost of 203 mm thick CRCP (excluding sub-base) was quoted as between $ 7.2 and $ 8.2 per m$^2$ and it was stated that this was less than the cost of 225 mm thick JCP. A contractor in Louisiana quoted prices per m$^2$ of $ 6.60 for 250 mm thick unreinforced concrete and $ 7.2 for 200 mm CRCP with 0.6 per cent steel and $ 8.4 to $ 9.00 for 225 mm thick CRCP. A similar cost for 250 mm thick JCP was quoted in Indiana where it was said that the cost of 225 mm thick CRCP was greater than this. In Texas a cost per m$^2$ of between $ 7.2 and $ 7.8 was quoted for 203 mm thick CRCP; no costs were mentioned for JCP. Some other comparative cost figures have been obtained from published reports and these are given in Appendix 1 Table 4. Generally it will be seen that the CRC roads are 15–20% more expensive than the conventional jointed pavements when considered only on the basis of initial costs.

The same difficulty in obtaining true comparative costs was experienced in Belgium. The only available cost figures from that country are quoted as 730 B Frs per m$^2$ for 230 mm thick jointed pavement (unreinforced) and 845 B Frs per m$^2$ for 200 mm thick CRCP (with 0.85 per cent steel), or 16 per cent more expensive. This again confirms the American experience that on first costs alone CRC roads cannot be justified.

An attempt has been made to predict the increased cost of a CRCP road over the unreinforced JCP which is currently in widespread use in Great Britain. This is based on calculating the savings effected by the use of a reduced thickness of concrete, and by the omission of contraction joints and the underlay;
savings have been offset against the cost of the reinforcement. The designs considered are for pavements to carry 11 million and 60 million standard axles in a design life of 40 years. The assumption made is that in each case the Road Note No 29 design thickness for JCP can be reduced by 30 mm when CRCP is used and that the steel percentage for the latter is 0.6. Appendix 1 Table 5 shows the design for both plain and reinforced JCP and for CRCP.

Based on recent billed prices for unreinforced JCP the savings by reducing the concrete thickness by 30 mm could range from £0.20 to £0.23 per m² and the savings brought about by the omission of contraction joints at 5 m spacings could range from £0.21 to £0.45 per m²; by omitting the underlay a saving of between £0.06 and £0.10 per m² could also accrue. This gives a total saving of between £0.47 and £0.78 per m². The cost of the reinforcing steel is estimated to be between £1.01 and £1.13 per m² for the 12.6 kg/m² material and between £0.82 and £0.92 per m² for the 10.2 kg/m² material. This indicates that for the very heavily trafficked road the increase in initial costs could be between £0.23 and £0.66 per m² and for the road carrying 11 msa the increase could be from £0.04 to £0.45 per m².

In the above calculations the steel costs (as at April 1973) have been based on the use of prefabricated mesh. The prices for loose steel rod are lower than those for mesh (for the same weight of steel) and while with loose bars there is an increase in labour costs it is possible that the use of such material might lead to a reduction in costs and hence in the increase in costs over JCP. In the USA several contractors have adapted slipform pavers to lay loose rods with seemingly successful results.

Preliminary calculations show that the cost of CRCP to the Study Groups recommendations is likely to be less than the cost of CRC base with bituminous surfacing to the Road Note No 29 recommendations.

13.2 Maintenance costs

One of the reasons proffered for the widespread use of CRCP in America is that this form of construction requires little or no maintenance. If properly constructed and with a suitable crack spacing (and hence crack width) a pavement constructed with continuous reinforcement should only require maintenance to the longitudinal joints, the joints at the terminals and to any minor spalling. Again unfortunately there is a lack of any detailed and long-term records of maintenance costs of CRCP. The only figure obtained from the USA is for the State of Illinois which quotes a reduction in maintenance cost for CRCP in comparison with JCP of $0.41 per m². In Belgium there is a three year maintenance period and this stipulation must obviously reflect in the bid price and also in the maintenance costs subsequent to opening to traffic.

Although in the USA it is often said that the total costs of a road, ie initial costs and maintenance costs, show the CRC pavement in a favourable position when compared with jointed pavements there is little factual evidence to support this. The Belgian source mentioned earlier has included some idealised maintenance costs for a period of 15 years and when these are added to the initial construction costs the comparative figures are 990 B Frs per m² for 230 mm thick JCP and 953 B Frs per m² for 200 mm thick CRCP. The same source also predicts that because of the wider use of CRCP the initial cost of this form of construction will fall by between 50 and 75 B Frs per m² which will make the CRCP even more competitive.

An interesting development noted in the USA is the inception of a research project by the Federal Highway Administration to investigate 'Premium pavements for zero maintenance'. Outlining this project mention is made of rough estimates which show that an additional half-million dollars per mile could be spent on the construction of a premium pavement on a high volume six lane urban expressway if this...
pavement would double the length of time necessary for the first overlay. This is roughly equivalent to an increase in the initial cost per square metre of £5.9.

13.2.1 Use of LR 256 to predict savings in maintenance costs Report LR 256 assessed the construction, maintenance and traffic delay costs of flexible and concrete roads. This was the first time that maintenance costs had been included in any cost analysis and, while many of the maintenance and delay costs were broad estimates because of lack of reliable data, the method of calculation was established as reasonable. In a CRC road pavement there should be no maintenance requirement for transverse joints; thus if the cost of maintaining transverse joint is excluded, a comparison can be made between the likely maintenance costs of CRCP and JCP assuming similar structural behaviour. This comparison has been made for total costs and also for ‘present values’ (ie with discounted maintenance and delay costs) for the four types of road considered in LR 256. These were i) a rural motorway (three-lane dual carriageway), ii) a peri-urban road (two-lane dual carriageway), iii) a rural secondary road (two-lane single carriageway) and iv) a housing estate road (two-lane single carriageway); the discount rate used in calculating ‘present values’ was 8 per cent per annum. The comparisons are made in Appendix 1 Tables 6 and 7.

It will be seen from these tables that the saving on total maintenance cost over a 50 year period is likely to be less than the additional construction costs estimated in section 13.1 of this appendix. When the maintenance costs are discounted to present day values they become generally insignificant. Delay costs to traffic are also small for those roads on which it is possible to carry out maintenance operations without undue traffic delays eg the three-lane rural motorway and the housing estate road. Where however any maintenance work will cause serious delays, as with the peri-urban two-lane dual carriageway, the delay costs rise steeply. It is under these latter conditions that the economic grounds for CRCP appear strongest.

14. ACKNOWLEDGEMENTS

The Study Group is very grateful to all the American and Belgian engineers who made the visits to their countries so successful.

15. REFERENCES


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**APPENDIX 1**

**TABLE 1**

Slab thicknesses (mm) for different designs

<table>
<thead>
<tr>
<th>Concrete type</th>
<th>C1</th>
<th>C2</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>ACI</td>
<td>CRPG</td>
</tr>
<tr>
<td>Design</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Soil S1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Traffic T1</td>
<td>300</td>
<td>180</td>
</tr>
<tr>
<td>T2</td>
<td>240</td>
<td>150</td>
</tr>
<tr>
<td>T3</td>
<td>150</td>
<td>110</td>
</tr>
<tr>
<td>T4</td>
<td>130</td>
<td>100</td>
</tr>
<tr>
<td>Soil S2</td>
<td>150 mm Sub-base</td>
<td>Granular</td>
</tr>
<tr>
<td>Traffic T1</td>
<td>T2</td>
<td>250</td>
</tr>
<tr>
<td>T3</td>
<td>160</td>
<td>150</td>
</tr>
<tr>
<td>T4</td>
<td>140</td>
<td>120</td>
</tr>
<tr>
<td>Soil S3</td>
<td>150 mm Sub-base</td>
<td>Granular</td>
</tr>
<tr>
<td>Traffic T1</td>
<td>T3</td>
<td>250</td>
</tr>
<tr>
<td>T3</td>
<td>170</td>
<td>160</td>
</tr>
<tr>
<td>T4</td>
<td>150</td>
<td>130</td>
</tr>
</tbody>
</table>

* These values are obtained by using the Nomograph (extrapolated for traffic T1)
# APPENDIX 1

## TABLE 2

Areas of steel (mm$^2$) per metre width for different designs

<table>
<thead>
<tr>
<th>Concrete type</th>
<th>C1</th>
<th>C2</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Design</strong></td>
<td><strong>ACI</strong></td>
<td><strong>CRPG</strong></td>
</tr>
<tr>
<td></td>
<td>Reinforced</td>
<td>Plain</td>
</tr>
<tr>
<td><strong>Steel percentage</strong></td>
<td>0.54</td>
<td>0.60</td>
</tr>
<tr>
<td><strong>Soil S1</strong></td>
<td>Traffic T1</td>
<td>1620</td>
</tr>
<tr>
<td></td>
<td>T2</td>
<td>1296</td>
</tr>
<tr>
<td></td>
<td>T3</td>
<td>810</td>
</tr>
<tr>
<td></td>
<td>T4</td>
<td>702</td>
</tr>
<tr>
<td><strong>Soil S2</strong></td>
<td>Traffic T1</td>
<td>1674</td>
</tr>
<tr>
<td></td>
<td>T2</td>
<td>1350</td>
</tr>
<tr>
<td></td>
<td>T3</td>
<td>864</td>
</tr>
<tr>
<td></td>
<td>T4</td>
<td>756</td>
</tr>
<tr>
<td><strong>Soil S3</strong></td>
<td>Traffic T1</td>
<td>1728</td>
</tr>
<tr>
<td></td>
<td>T2</td>
<td>1350</td>
</tr>
<tr>
<td></td>
<td>T3</td>
<td>918</td>
</tr>
<tr>
<td></td>
<td>T4</td>
<td>810</td>
</tr>
</tbody>
</table>

* These values are obtained by using the Nomograph (extrapolated for traffic T1)
### APPENDIX 1

#### TABLE 3

Weights of steel (Kg/m²) for different designs (1.4 Kg/m² assumed for transverse steel)

British weights are for BS 4483 standard mesh fabrics

<table>
<thead>
<tr>
<th>Concrete type</th>
<th>C1</th>
<th>C2</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>ACI</td>
<td>CRPG</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Steel percentage</td>
<td>0.54</td>
<td>0.60</td>
</tr>
<tr>
<td>Soil S1 Traffic T1</td>
<td>14.1</td>
<td>9.9</td>
</tr>
<tr>
<td>T2</td>
<td>11.6</td>
<td>8.5</td>
</tr>
<tr>
<td>T3</td>
<td>7.8</td>
<td>6.6</td>
</tr>
<tr>
<td>T4</td>
<td>6.9</td>
<td>6.1</td>
</tr>
<tr>
<td>Soil S2 Traffic T1</td>
<td>150 mm</td>
<td>Sub-base</td>
</tr>
<tr>
<td>Traffic T1</td>
<td>14.6</td>
<td>14.1</td>
</tr>
<tr>
<td>T2</td>
<td>12.0</td>
<td>11.1</td>
</tr>
<tr>
<td>T3</td>
<td>8.2</td>
<td>7.8</td>
</tr>
<tr>
<td>T4</td>
<td>7.3</td>
<td>6.5</td>
</tr>
<tr>
<td>Soil S3 Traffic T1</td>
<td>150 mm</td>
<td>Sub-base</td>
</tr>
<tr>
<td>Traffic T1</td>
<td>15.0</td>
<td>14.6</td>
</tr>
<tr>
<td>T2</td>
<td>12.0</td>
<td>11.6</td>
</tr>
<tr>
<td>T3</td>
<td>8.6</td>
<td>8.2</td>
</tr>
<tr>
<td>T4</td>
<td>7.8</td>
<td>6.9</td>
</tr>
</tbody>
</table>
### APPENDIX 1

#### TABLE 4

Some comparative cost figures between CRCP and JCP

<table>
<thead>
<tr>
<th>State</th>
<th>Year</th>
<th>JCP</th>
<th>CRCP*</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Cost of paving $ per m²</td>
<td></td>
</tr>
<tr>
<td>Oregon</td>
<td>1962</td>
<td>203mm slab 5.37</td>
<td>203mm slab 6.46</td>
</tr>
<tr>
<td></td>
<td>1962</td>
<td>&quot; &quot; 6.57</td>
<td>&quot; &quot; ( &quot; &quot; ) 7.76</td>
</tr>
<tr>
<td>Pennsylvania</td>
<td>1957</td>
<td>254mm slab 6.28</td>
<td>178mm slab 5.53</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.5% steel (bar mat)</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>203mm slab 5.70</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.5% steel ( &quot; ) 5.41</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>229mm slab 5.62</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.5% steel ( &quot; ) 5.30</td>
<td></td>
</tr>
<tr>
<td>Maryland</td>
<td>1958</td>
<td>229mm slab 7.19</td>
<td>203mm slab 7.19</td>
</tr>
<tr>
<td></td>
<td>1960</td>
<td>&quot; &quot; 6.02</td>
<td>203mm slab 6.80</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Joints at 12.2m</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.6% steel (bar mat)</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>203mm slab 6.20</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.6% steel (fabric)</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>203mm slab 6.61</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.6% steel (fabric)</td>
<td></td>
</tr>
<tr>
<td>Minnesota</td>
<td>1963</td>
<td>229mm slab 5.30</td>
<td>203mm slab 5.89</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Joints at 12.0m 152mm sub-base</td>
<td>152mm sub-base 6.20</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.5% steel (fabric)</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>203mm slab 6.20</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.6% steel (fabric)</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>203mm slab 6.45</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.7% steel (fabric)</td>
<td></td>
</tr>
</tbody>
</table>

* costs for CRCP normally exclude the cost of the terminal abutments
**APPENDIX 1**

**TABLE 5**

Comparison of CRCP and JCP designs for roads constructed on normal subgrade (recommended design life 40 years)

| Slab thickness (mm) | Traffic — cumulative number of standard axles | | |
|---------------------|---------------------------------------------|-----------------|-----------------|-----------------|-----------------|
|                     | 11 x 10^6                                  | 60 x 10^6       | 11 x 10^6       | 60 x 10^6       | 11 x 10^6       | 60 x 10^6       |
|                     | JCP Reinforced Unreinforced                 | JCP Reinforced Unreinforced | JCP Reinforced Unreinforced | JCP Reinforced Unreinforced | JCP Reinforced Unreinforced | JCP Reinforced Unreinforced |
| Slab thickness (mm) | 220                                         | 190             | 270             | 270             | 240             | 240             |
| Reinforcement:      |                                             |                 |                 |                 |                 |                 |
| Area of longitudinal steel (mm²/m width) | 420                                         | 590             | 1440            | 1440            | 1440            | 1440            |
| Weight (kg/m²)      | 4.34*                                       | 10.22           | 5.55*           | 5.55*           | 12.57           | 12.57           |
| including transverse steel |                |                 |                 |                 |                 |                 |
| Maximum spacing of contraction joints (m) | 27.5                                        | 35.0            | 5.0             | 5.0             | 5.0             | 5.0             |

* Based on the use of long mesh fabric reinforcement to BS 1221.
APPENDIX 1

TABLE 6

Savings in total costs possible by omission of transverse joints

<table>
<thead>
<tr>
<th>Road type</th>
<th>Years</th>
<th>Savings (£)</th>
<th></th>
<th></th>
<th></th>
<th></th>
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</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>per km length</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Mtce</td>
<td>Delay</td>
<td>Total</td>
<td>Mtce</td>
<td>Delay</td>
</tr>
<tr>
<td>Rural motorway</td>
<td>10</td>
<td>658</td>
<td>89</td>
<td>748</td>
<td>0.029</td>
<td>0.004</td>
</tr>
<tr>
<td>(hot poured joints)</td>
<td>20</td>
<td>1646</td>
<td>388</td>
<td>2033</td>
<td>0.073</td>
<td>0.017</td>
</tr>
<tr>
<td></td>
<td>30</td>
<td>2304</td>
<td>673</td>
<td>2977</td>
<td>0.102</td>
<td>0.030</td>
</tr>
<tr>
<td></td>
<td>40</td>
<td>3291</td>
<td>1269</td>
<td>4561</td>
<td>0.146</td>
<td>0.056</td>
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<tr>
<td></td>
<td>50</td>
<td>3950</td>
<td>1787</td>
<td>5737</td>
<td>0.175</td>
<td>0.079</td>
</tr>
<tr>
<td>Rural motorway</td>
<td>10</td>
<td>1863</td>
<td>52</td>
<td>1915</td>
<td>0.083</td>
<td>0.002</td>
</tr>
<tr>
<td>(cold poured joints)</td>
<td>20</td>
<td>3726</td>
<td>142</td>
<td>3868</td>
<td>0.165</td>
<td>0.006</td>
</tr>
<tr>
<td></td>
<td>30</td>
<td>5589</td>
<td>289</td>
<td>5878</td>
<td>0.248</td>
<td>0.013</td>
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<tr>
<td></td>
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## APPENDIX 1

### TABLE 7

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Appendix 1  Fig. 1  THEORETICAL RELATIONS BETWEEN SLAB THICKNESSES GIVING SAME STRESS FOR EDGE AND CORNER LOADINGS IN TERMS OF INTERIOR LOADING (TYRE PRESSURE 60 lb/in^2)
Appendix 1  Fig 2 THEORETICAL RELATIONS BETWEEN SLAB THICKNESSES GIVING SAME STRESS FOR EDGE AND CORNER LOADINGS IN TERMS OF INTERIOR LOADING (TYRE PRESSURE 120 lb/in²)
51mm Groove, hot poured seal
Bond breaker
CRCP
6 mm Groove, hot poured seal
Heavily greased
225 or 250 mm Pavement
Expansion joints
Dowels
Bridge

Expansion joint* material
3.05 m
1.52 m Taper
150 mm

X = 9.2 to 15.2 m JCP
Y = Standard bridge approach slab
* 25 mm Expansion joint for summer construction of CRCP otherwise 50 mm

Appendix 1 Fig. 3 BURDELL WIDE FLANGE BEAM JOINT FOR CRCP

3 or more 19mm expansion joints

PLAN OF TRANSVERSE LUGS

89 mm 229 mm 108 mm 108 mm
1.83 m
1.22 m
0.61 m

Construction Key
108 x 203 mm

SECTION A – A

Appendix 1 Fig 4 LUG TYPE TERMINAL ANCHOR

Adjacent pavement
19 mm Expansion joint
Same thickness as adjacent pavement
Appendix 1 Fig. 5 BELGIAN PRACTICE — ARRANGEMENT AT BRIDGES WITH DEPTH OF FILL GREATER THAN 1m
Appendix 1 Fig. 6 BELGIAN PRACTICE — ARRANGEMENT AT BRIDGES WITH DEPTH OF FILL LESS THAN 1M
Appendix 1 Fig. 7  BELGIAN PRACTICE – ARRANGEMENT AT BRIDGES WHEN PAVEMENT IS INTERRUPTED
16. APPENDIX 2

16.1 Lists of American engineers met by the Study Group

16.1.1 University of Texas at Austin
   Dr W R Hudson
   Dr B F McCullough
   Mr A Abou-Ayyash

16.1.2 Texas State Highway Department, Austin
   Mr R L Lewis
   Mr G B Peck
   Mr L Butler

16.1.3 Texas State Highway Department, Houston Expressway Office
   Mr W V Ward

16.1.4 Louisiana Department of Highways, New Orleans
   Mr Graves
   Mr Taylor
   Mr Inabinet (Federal Highway Administration)
   Mr Williams (T L James & Company, Inc)
   Mr James ("")
   Mr Westbrook ("")
   Mr Kuhlman (Portland Cement Association)
   Mr Lambert (Lambert Construction Co)

16.1.5 Mississippi State Highway Department, Jackson
   Mr A M White
   Mr J P Steinwinder
   Mr R W Thomas
   Mr H V Mahan
   Mr T C Teng

16.1.6 Continuously Reinforced Pavement Group, Chicago
   Mr J D Lindsay
Mr E Lokken (Portland Cement Association)
Mr Scottkovsky (O'Hare Airport, Chicago)

16.1.7 Portland Cement Association, Skokie, Illinois
Mr G K Ray
Mr Ross Martin
Mr E Lokken
Mr B E Colley

16.1.8 Indiana State Highway Commission, Indianapolis
Mr G K Hallock
Mr S Yoder
Mr R Roath
Mr A Livingston

16.1.9 Maryland State Highway Administration, Baltimore
Mr D H Fisher
Mr N L Smith
Mr F S Kinney
Mr C E Starkey
Mr W Lins
Mr W Jahreis
Mr J D Spencer
Mr B Sedgewick

16.1.10 Federal Highway Administration, Fairbanks Research Station, Washington, D.C.
Mr E C Wiles
Dr T F McMahon
Mr T J Pasko
Mr McComb

16.1.11 Federal Highway Administration, Department of Transportation, Washington, D.C.
Mr E H Karrer
Mr Smagala
Mr Kayser
Mr C V Smith
Mr F J Geiser, Jr
Mr J W Hewett
Mr W H Carter
Mr J C Kliethermes
Mr E E Rugenstein

16.2 CRCP roads visited by the Study Group

16.2.1 Texas

I 10 Western end to Houston

16.2.2 Houston, Texas

I 10 Katy Freeway
I 45 Gulf Freeway

16.2.3 Louisiana

I 10
I 55

16.2.4 Mississippi

I 55

16.2.5 Chicago area

I 90 Dan Ryan Expressway
I 55 Stevenson Expressway
O'Hare Airport
Midway Airport

16.2.6 Indiana

I 65
US 40 at Stilesville
16.2.7 Maryland

195
183
17. APPENDIX 3

BELGIAN VISITS BY THE STUDY GROUP

Two short visits were made to Belgium by members of the Study Group as follows:

June 29 - 30    Mr A E Burks
               Mr J M Gregory
               Mr J R Lake

October 2 - 4    Mr A E Burks
                 Mr J M Gregory

The following Belgian engineers were met during the course of these visits:

M A Doyen (Ministere des Travaux Publics, Administration des Routes)
M J Chavet " "
M A Van Loocke " "
M P Dutron (Centre National de Recherches Scientifiques et Techniques pour l'Industrie Cimentiere)
M P Van Ael " "
M Y Dechamps " "

CRCP roads visited in Belgium

E 5 Brussels — Liege
N 5 Laneffe — Somzee
N 5 Frasnes-lez-Gosselies — Mellet
E 3 Ghent — Courtrai
A 8 Hertain — Lamain
N 61 Barry — Bury
N 8 Leuze
Boulevard de Borinage
E 41 Liege — Namur — Velaine
N 485 Andenne — Bierwart
Fig. 1  DEVELOPMENT OF CRCP IN USA
PLATE 1

Typical crack pattern of CRCP
(USA - I 10 New Orleans)
PLATE 3

Experimental CRCP road in Belgium after 22 years (N 8 at Leuze – Faulted crack in section with 0.3 per cent steel)
Experimental CRCP road in Belgium after 22 years (N 8 at Leuze Foreground is section with 0.5 per cent steel, background has 0.3 per cent steel).
PLATE 5

Laying of prefabricated reinforcement mats
(Belgium A 8 Hertain — Lamain. The mats have supports welded to the transverse reinforcement)
PLATE 6

Laying CRCP with conventional concreting train
(Belgium E 3 Ghent — Courtrai. Longitudinal wires placed
into clips which are part of supports welded to transverse
reinforcement)
PLATE 7

Laying CRCP with slipform paver
(Belgium E 5 Brussels — Liege. The reinforcement is laid well ahead of paving operations. Slipform paver is side fed from other carriageway.)
ABSTRACT

Continuously reinforced concrete pavements a report of the Study Group: J M GREGORY, Pavement Design Division, TRRL, A E BURKS, Consulting Engineer and A PINK, Cement and Concrete Association: Department of the Environment, TRRL Report LR 612: Crowthorne, 1974 (Transport and Road Research Laboratory). Continuously reinforced concrete pavements are widely used in the USA and also in Belgium. A Study Group of British engineers has visited both these countries to study the design, construction and performance of this type of pavement.

The concept of continuously reinforced concrete pavements is that cracks will occur at intervals of between 1.5 and 2.5 metres but that these cracks will be kept tightly closed by the reinforcement thus maintaining load transfer across the cracks by aggregate interlock.

No special construction equipment is necessary for continuously reinforced concrete pavements and there is no restriction on length other than a minimum of 150 m. The Study Group have concluded that for heavily trafficked roads the slab thickness for a continuously reinforced concrete pavement may be 30 mm less than that required for a jointed concrete pavement; this is associated with a steel percentage of 0.6.

The initial cost of a continuously reinforced concrete pavement is likely to be greater than that of a jointed concrete pavement for the same traffic conditions. Maintenance requirements are less and for heavily trafficked urban roads where delay costs due to maintenance operations may be high there could be economic advantages in the use of continuously reinforced concrete pavements.